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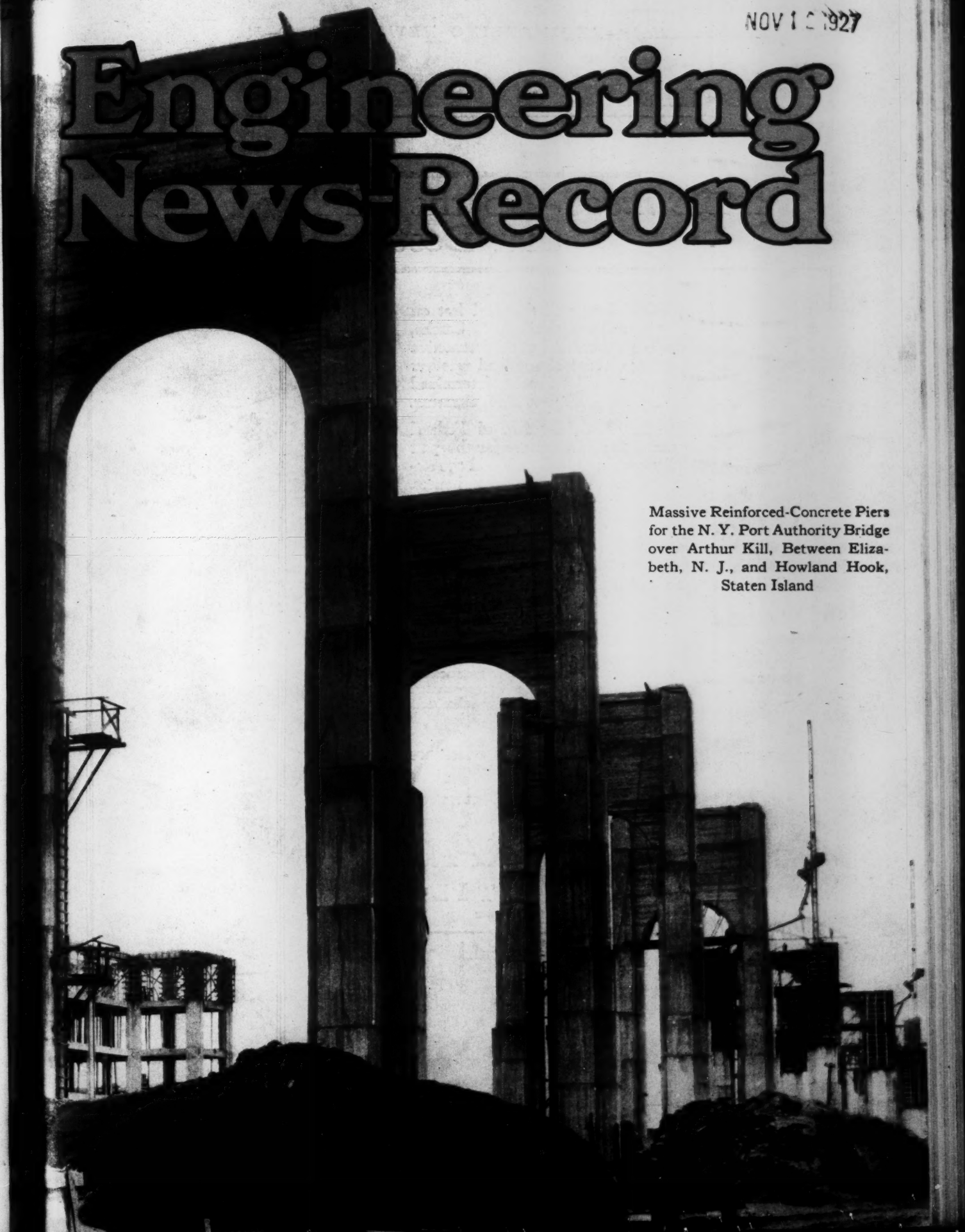
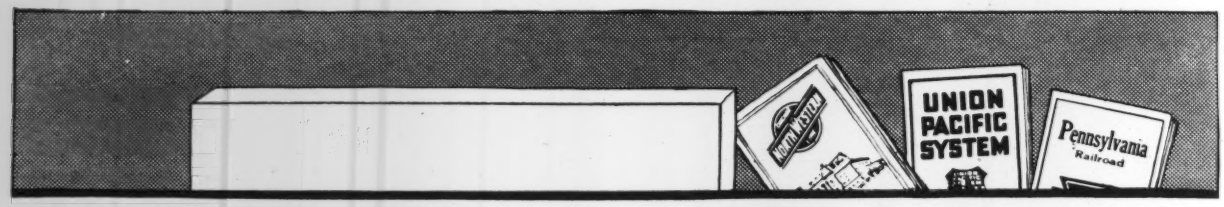
Engineering News-Record



Massive Reinforced-Concrete Piers for the N. Y. Port Authority Bridge over Arthur Kill, Between Elizabeth, N. J., and Howland Hook, Staten Island



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ENGINEERING NEWS-RECORD

DEVOTED TO CIVIL ENGINEERING
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Flood Dangers Unforeseen

ON PRESENT information the New England floods of the past week exceeded all records in those regions for at least half a century. There had been similar extreme stages of the rivers long ago, but they had been forgotten, or nobody believed that they would recur. Except for the particular period of time involved, this is a precise repetition of the circumstances surrounding all of the dozen or score of other floods that have visited various regions of the United States in the past decade. Thus the oldest settled part of the country now finds itself in the same situation of unpreparedness against floods as the younger regions—the central states, the Great Plains and the mountain country. Nowhere have rainfall and flood contingencies been made a matter of systematic study until after a great flood had wrought devastation. So long as this condition remains the danger will continue to be ignored and great flood disasters will feature the day's news once or twice a year. The bitter experiences of the extreme floods are easily forgotten during the years of quiet, and neither communities nor individuals will remember to respect flood channels or will undertake to build works that might control or carry off the runoff of great storms. But the failure to make flood studies is not to be laid up against local communities. The problem is of nation-wide scope, and only the federal government is equipped to take it in hand. Hitherto the government's scientific bureaus have made only a small, hesitant beginning toward collecting and analyzing the complex data of such a study. But the present year, with the great Mississippi flood followed by the New England flood, presents an imperative demand that the work be undertaken at once, on broadest possible scale. The Congress soon to meet will have no more urgent responsibility upon it than to provide funds for the study.

An A.G.C. Safety Manual

DEFINITE evidence of activity by the Associated General Contractors in their campaign of accident prevention is furnished by the manual just published by that organization. Stating the doctrine of accident prevention which this body of general contractors holds fundamental, the code of safe practices on construction work employed successfully by the Detroit chapter of the A.G.C. since 1921 is offered as a basis of practice. How this specific group of rules meets legal restrictions and general construction conditions in the wide territory of 48 states may be a subject for dispute, but for the moment this question may be neglected to make place for the consideration of the broader question of the association's policy of accident prevention. Briefly, this is that constructors should so set their house in order and institute for themselves such practically effective measures of protecting their work and their workmen, that other agencies are not compelled to repair their neglect. There is sound common sense in this attitude. Either the contracting business must discipline itself in methods of safety, or this discipline will be thrust upon it by organized labor. Anyone who has observed the trend of action and pronouncement by the workmen's body in the last year or two must come to this conclusion, and there is no wisdom in mincing words in expressing it. If organized contracting does not codify and enforce safety practices, organized labor will do it. Mandatory safety requirements formulated by others promise to be a burden on construction beyond the good they will accomplish. In stressing the doctrine of self-regulation in safety work, the manual of the Associated General Contractors performs a wise action. It takes a wise stand, too, in emphasizing the possibilities of group accident-prevention work to meet the many situations where individual constructors are not able to undertake effective education and regulation by themselves, but can obtain both by acting in groups and sharing costs. Such group action has been worked out successfully in a number of localities.

Good Building

THE substructure work for the two Arthur Kill bridges described in this issue proved an excellent test of the versatility of the engineers and contractors who had to design and carry out the work in the field. For this reason a rather circumstantial account of the operations has been written. No summary here is contemplated. The fact stands out, however, that in all of the scores of piers constructed, only small groups, the units of which were often scattered, were alike in foundation conditions. There was therefore continual need for change of design by the engineers and change of construction procedure by the contractors. Somewhat remarkably at the same time the work, consisting of many similar structures, encouraged ambition to standardize on equipment and processes. There was some such standardization. The use of traveling mixers and of light mast hoists and gin poles for placing concrete is perhaps the best illustration. On the whole, however, the pier building, like the foundation excavation and construction, was noted for the freedom exercised in changing and adapting plant and procedure to meet the immediate conditions. An example of this liberty in making changes is furnished by the substitution by one contractor, with the engineers' consent, of structural frame reinforcement for the planned loose rod reinforcement. It met the design requirements as well as did the original arrangement of rods and it was, therefore, permitted by the engineers because it offered advantages of economic construction to the contractor. By putting a little more money into the skeleton of reinforcing steel he provided a frame to carry the forms and a staging by which to

swing them into place. All through the work like co-ordination and continuity of purpose in design and construction come into evidence in reading the details of procedures followed. This is good building.

Fire in Vehicle Tunnels

WHAT would happen in the case of a serious fire in a vehicle tunnel has been the subject of considerable discussion in recent months, particularly in reference to the Holland Tunnel at New York. Would a burning automobile become a menace to the people in the tunnel? Would the ventilating system be able to take off the large quantity of smoke generated, or would it spread through the tunnel, causing a panic, if not actually endangering life? These questions have been answered for the Holland Tunnel as well as is possible short of having a fire occur in traffic after the tunnel is opened. An automobile, so saturated with gasoline as to make a much worse blaze than would occur under normal conditions, was set on fire in the tunnel last Thursday. At the height of the fire, when flames leapt up against the ceiling, there was scarcely a noticeable amount of smoke down in the roadway where the occupants of automobiles close to the fire would be located under actual conditions. Even close to the car, the supply of fresh air blowing in at both sides of the roadway kept the atmosphere clear. The black clouds of smoke spread along the ceiling for a considerable distance each side of the fire but high enough to be well above any one in the roadway or on the sidewalk. These clouds disappeared rapidly. They were gone before the three and one-half minutes required to put out the fire entirely had elapsed, leaving nothing but a faint haziness along the upper part of the tunnel. The longitudinal draft which will be induced by traffic was not present during the test and consequently it is not possible to foretell exactly what would happen with many cars in the tunnel, nor can it be foretold what would happen under similar conditions if a gas tank exploded and spread fire to adjoining cars, but judging by conditions in the tunnel during the fire test there is little reason to believe that such a catastrophe, which would be serious even on the open road, would result in a panic. The bright illumination of the tunnel and the clearness of the atmosphere in the roadway despite the dense smoke overhead did not tend to create such a state of mind.

Progress in Water Treatment

ADVANCES in the art of water purification during the past two or three decades are epitomized elsewhere in this issue in an article designed primarily to show progressive changes in water treatment made by the Indianapolis Water Company. What has been done there is so well co-ordinated with what has been going on elsewhere as to give a good general picture of the progress of the art in America. As was the case in many other cities in the nineties and early 1900's, large, open slow sand filters rather than rapid were chosen for the first plant at Indianapolis. Soon these were divided into smaller units and covered. Then pre-sedimentation, with provision for coagulation, was added. Then came the use of chlorination. These changes increased the capacity and efficiency of the old plant. When still more capacity was needed mechanical filters were added, in accordance with the general practice of today.

Ice Thrust Against Dams

ICE thrust against dams continues to be a subject upon which there is little definite knowledge. What small amount of knowledge we have is not sufficiently conclusive to be used as a broad basis in design. It serves simply as a guide. Fuller studies are needed.

Ice thrust may develop from two causes: Expansion of ice in process of formation within a confined area, and the thrust of a thick sheet of ice as it expands due to temperature increases within a range below the freezing point. While the first of these is capable of demonstration under laboratory conditions, it is not likely to have practical importance. The second, or temperature-expansion thrust, is the factor which the designers of dams must consider. It may be a serious menace, particularly to small dams. A thick sheet of ice expanding under a considerable rise in temperature will increase appreciably in area. Such a sheet a mile long will increase in length approximately 1½ ft. for each 10-deg. rise in temperature. With the frequent temperature changes below the freezing point which occur during a northern winter this force may develop into a greater menace than is at first apparent. The under surface of a sheet of ice, being in contact with water, remains approximately at the freezing point, while the top surface (if not heavily blanketed with snow) may be cooled off by the air far below the freezing point. When the ice temperature decreases shrinkage cracks develop which immediately fill with water and freeze.

By such a process there is built up a potential thrust in the ice sheet with possibilities for danger to light structures in the path of its expansion. In a truly confined area, this movement may develop great forces, but where the shores have an easy slope the ice sheet may shove up the slopes and dissipate all or most of its energy. Where the sheet covers a small forebay between a dam and a vertical rock wall, the expansion is almost certain to affect the dam seriously if the sheet becomes very thick.

As a practical matter, it is probable that stresses calculated for ice expansion will rarely be in full effect. There is some doubt as to how solid the ice forming in expansion cracks becomes; also, buckling is not uncommon even in very thick ice sheets. Nevertheless, the magnitude of possible ice effects cannot be ignored in climates where temperatures well below freezing are common for any considerable length of time.

Early attempts to relieve the increasing menace of a thickening ice sheet pressing against dams and headworks included removal of enough ice to maintain a narrow open channel just in front of the upstream face of the structure wherein expansion could be taken up. This was effective, but slow and expensive. Steam was then tried, with much the same result and objection. A more satisfactory means of avoiding the danger altogether was finally found in the release of compressed air under water so that a row of air bubbles continually would rise to the surface along the line where the open water is desired. As these bubbles break on the surface they cause a radial flow of water, whose temperature is above freezing, to pass the edges of the adjacent ice sheet and thus check the progressive action of freezing up the open channel.

Ice sheets pushed up the face of sloping dams by expansion due to temperature increase suggest the possibility of building dams that would have inherent in their design a protection against possible ice thrust. At stor-

age dams in remote regions, for example, where it is not necessary to keep an attendant in winter, the compressed-air method is not a feasible means of protection against ice thrust. In such cases it might be well worth a considerably greater first cost to obtain a structure that is not subject to ice dangers.

Some evidence as to the force of ice thrust has been collected at Keokuk in a gate structure that was broken by the thrust of an expanding ice sheet. This thrust, even though partially relieved by round-nosed piers above the broken gate, exceeded 3.7 tons per linear foot. Investigations conducted in Norway, as reported in *Engineering News-Record* of Aug. 14, 1924, p. 266, resulted in a conclusion that the maximum thrust from temperature ranges likely to occur in that country will not exceed 6 tons per linear foot for a 30-in. ice sheet, or 9 tons per linear foot for a 40-in. sheet.

Upstream faces that have as much slope as is commonly used on hollow dams (about 1 to 1), or even the steeper slopes found on multiple-arch dams, would probably not be damaged by ice thrust. Dams of this type, however, have the inherent disadvantage in cold climates that the cold may penetrate the thin structure and cause freezing on the upstream face. Solid dams, which are therefore preferred, cannot well be built with sloping upstream face on account of prohibitive cost.

These are the difficulties faced by those who would design dams for use in cold climates. Experience and experimental studies carried on heretofore are sufficient to indicate how real they are. Further studies as to the amount of ice thrust, suggested in a recent discussion before the American Society of Civil Engineers, would contribute greatly toward placing the question on a more definite basis.

Lighting and Illumination

NOT many weeks ago a reduction in the price of incandescent lamps was announced, a further step in a long history of progress that has, if memory serves correctly, made the highly efficient lamp of today cost barely a tenth as much as the inefficient lamp of a generation ago. The news means that human life is to be lifted to a higher level, for modern civilization is to a significant degree dependent upon the lamp-making art. Were it not for the electric lamp, the ubiquitous wire lines would be incomparably more rare, and most of our power houses would yet be undreamed of; it was the lamp that brought about the development of large-scale power and made possible its transmission to every nook and corner of existence. And so the latest extension of the possibilities of lighting, expressed in lower prices for lamps, means more power plants, remoter transmission of power, and further transfer of the world's heavy work from muscle to machine. It will increase the productivity of man's labors and give him more leisure to utilize and enjoy his time on earth.

In another aspect the announcement is less happy. It should mean that we will not only use more light, but will also have more effective illumination. But unfortunately this will not be the case. In spite of the incomparable resources of electric current and high-efficiency lamps which we command today, our actual light utilization is woefully inefficient. Illumination is one of the most amply endowed and yet most backward of the engineering arts. Its shortcomings, unfortunately, affect every man's comfort and wellbeing.

Modern man exists ever more by the strength of his eyes. The demand upon vision increases daily. Though we have always had to guide our footsteps by aid of the eye, today this process has reached such a refinement that the most minute failure in acuteness or quickness of sight, or the slightest delay in the response of hand to eye, may spell injury or death. Reading—as a necessity—has become universal and intense, and we must read by night as well as by day. Accordingly, the world has been flooded with light. Since the fragile little globe of glass with its magic filament was first created for us, it has wrought a transformation by virtue of which darkness is little more than a relative term. We have abundant light indeed, and we continue to provide more of it each day; but how efficiently are we utilizing those prodigious quantities of light-energy with which we stir the ether? The answer is discouraging.

Everywhere the eye is met by painful brilliance of lamps, the result of lavishing candlepower by the untold millions on the mission of making light; but of these many candlepowers, only a few are effective in lighting up that which ought to be seen. The lamps obtrude themselves upon us and monopolize vision, while only a negligible quantity of light comes to us as dim reflection from other objects—from room or from road—leaving these objects almost dark by comparison. In home or office, street or assembly place, we must content ourselves with indifferent illumination, obscured by contrast with brilliant masses of direct light.

It is true that compared with the tallow-candle days we have today more illumination as well as more light. But in proportion to the tremendous light contrasts that we create, illumination is perhaps even less efficient than in that primitive period. When we light our streets with powerful balls and clusters of glare, we leave pavement and curb line in an uncertain murk. When we light our homes and offices brilliantly we make reading less easy and less comfortable. The electric sign and the automobile headlight are no doubt the outstanding examples of misdirected use of light, but they are merely typical. Thus, the art of utilizing the wonderful means of light production placed in our hands by science and technology is still in the dark ages.

Civil engineers even more than other men are concerned in the backwardness of illumination, for the effectiveness of many of their plans and works is involved. They may construct office buildings or factories after careful study to produce efficient working places, only to find later that bad illumination—glare and ill-distributed light and shade—makes the workers inefficient. They may plan streets for maximum safety and usability, but must contemplate with dismay the obtrusively brilliant skyline and deceptively dark street surface which results.

The goal of replacing darkness by artificial daylight is still far off. But progress comes in response to human needs. We sadly need illumination; no doubt we will achieve it. Perhaps, too, the newest field of lighting needs, the road, will be one of the instruments of this progress. Development is urging strongly toward the lighted road, and as the fruitlessness of mere road lighting becomes clearly apparent, we may move forward toward the illuminated road. We may advance even farther, and obtain a headlight that illuminates instead of dazzles. And when so much progress has been made, possibly we may even be relieved of that antithesis of illumination, the electric sign. The future, we hope, is to be an age of illumination, not of lighting.

Airmap Corp. of America

ELIZABETH BRIDGE CROSSING OF ARTHUR KILL LOOKING TOWARD STATEN ISLAND

Building Substructure of Arthur Kill Bridges

Variable Pier Sinking Methods—Paving Mixers, Cranes and Mast Elevators Concrete 155 Piers Spread Over 15,000 Ft.—A System of Structural Steel Reinforcement

SUBSTRUCTURE WORK on the approaches to the two bridges which the Port of New York Authority is building over Arthur Kill between Staten Island and New Jersey has been notable in three main particulars. Varying soil and subsurface conditions have required an unusual diversity of pier foundation sinking methods, covering in their range unsheeted excavation, open cofferdams, both steel and wood sheeted, and a pneumatic caisson. For pouring the majority of the 155 approach piers, composed each of two shafts connected by an arch and ranging in height up to nearly 130 ft., mobile concreting plants of cranes, paving mixers and elevator masts have been efficiently adapted. Finally, on one contract, and in part on another, a system of structural steel pier reinforcement was used, while all other piers were reinforced by bars as originally planned.

The bridges, which are highway toll structures and whose general characteristics were described in *Engineering News-Record*, Aug. 26, 1926, p. 346, are known respectively as the Outerbridge Crossing, connecting Tottenville, Staten Island, with Perth Amboy, N. J., and the Elizabeth bridge connecting the Howland Hook section of Staten Island with Elizabeth, N. J. The main spans are cantilever truss structures, while the approaches are deck plate girders. In Fig. 1 are shown the locations of the bridges, while in Fig. 2 are given the principal dimensions.

Engineering Considerations—The contracts for the four approaches were let to three contractors, one of whom built both of the Staten Island approaches. Each contractor's task was essentially the same—to erect a series of arched reinforced-concrete piers, distributed over lengths varying from 3,070 ft. on the Elizabeth approach to 4,698 ft. on the Perth Amboy approach. In carrying out the tasks, however, different construction methods were used, caused largely by variations in foundation conditions.

Borings taken at the main channel pier on the Howland Hook approach (*C* in Fig. 2) indicated layers of broken shale, sandstone, soft clay and other material that failed to core. Because of the uncertain character and depth of this material, it was necessary to use a pneumatic caisson in constructing this pier, and to carry the excavation to rock at El. —69. Somewhat similar conditions were found at the anchor pier (*D*), but here the contractor elected to use an open cofferdam, since a dense clay formation sloping up from the channel made the water-

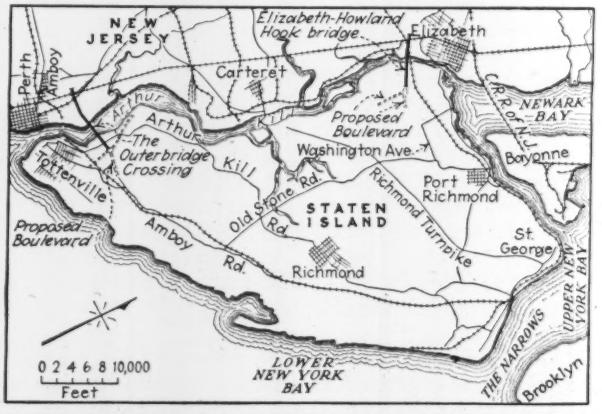

FIG. 1—LOCATION MAP FOR TWO ARTHUR KILL BRIDGES

handling problem not serious. For most of the piers on the Howland Hook approach, wood-pile foundations were necessary, and the excavation was carried only deep enough into the silt, and in some cases red clay, to give the necessary lateral support to these piers.

A few borings taken on the Elizabeth approach indicated rock so near the surface that the specifications called for all piers to rest on rock. However, extreme variability in rock depth was found by the contractor, and since he had then started to use ordinary 2-in. wood sheeting for the shallow depth expected, he continued with it until rock appeared. The first hole was sunk so successfully that wood sheeting was used on all the other approach piers, and this light sheeting of holes, as deep as 50 ft., was one of the important characteristics of this contract. For the channel pier on this approach, it was possible to use an open cofferdam since rock was fairly near the surface. It was, however, necessary to carry this pier about 22 ft. into the rock to provide against a future deepening of the ship channel.

the channel-bottom slit could be dredged easily, the open cofferdam method of construction was adopted.

The subsurface conditions on the Perth Amboy approach were very unfavorable, the silt formation, which continued inland for a distance of about 1,000 ft., containing much decayed vegetable matter. This material looked dry, but when disturbed developed into a jelly-like mud or gumbo. It would stand in almost vertical banks, yet exerted heavy pressure on the sheeting; closely braced steel-pile cofferdams were therefore necessary for piers *B* and *A*, and from 1 to 7. Pile foundations were necessary on a number of the piers, and considerable ground water made the use of wood piling possible on the two main piers and on the first nine approach piers. On 13 of the other piers near the center of the work,

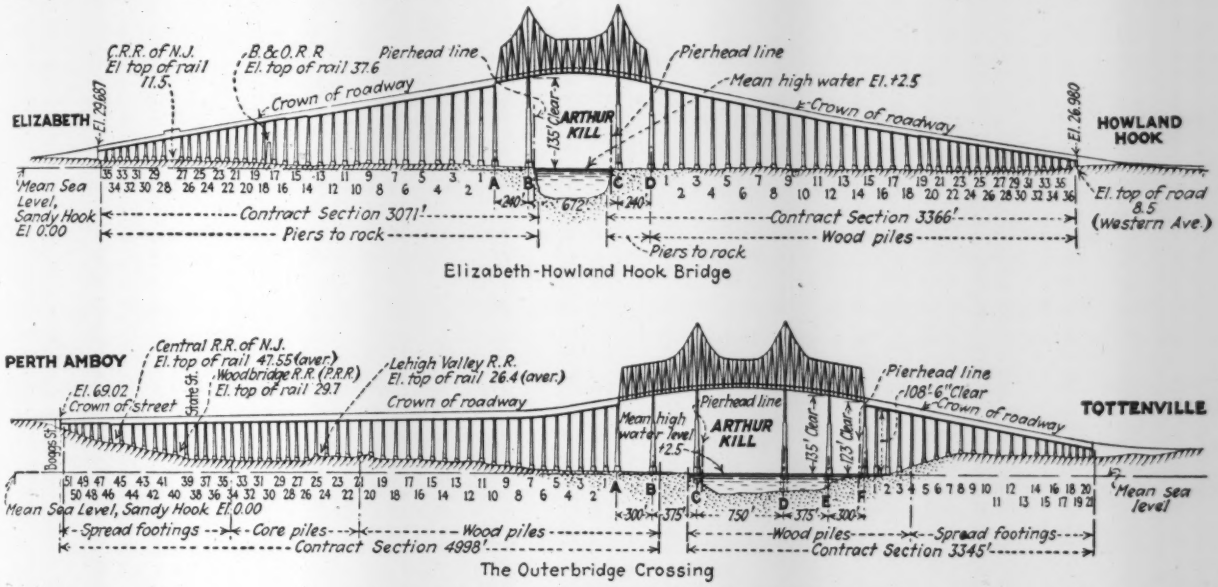

FIG. 2—PROFILES OF ARTHUR KILL BRIDGES

On the Tottenville approach, a firm clay-sand base fairly close to the surface made possible the use of spread footings on all piers except the four nearest the water. Here, pile supports were necessary, as they were on the four channel piers (*C, D, E* and *F*), where the subsoil was coarse sand and boulders of unknown thickness. Since

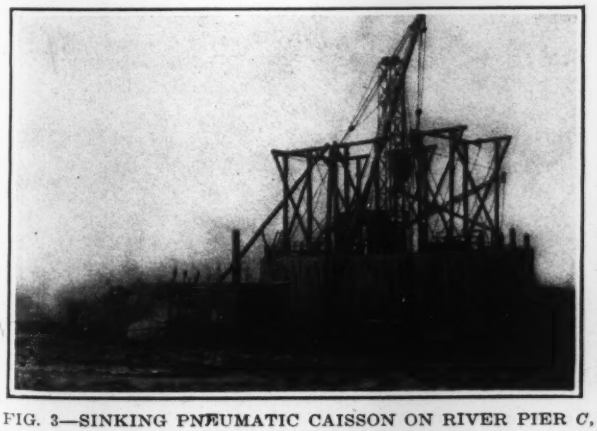

FIG. 3—SINKING PNEUMATIC CAISSON ON RIVER PIER *C*, HOWLAND HOOK

Note three telpher I-beams for transferring spoil buckets from locks to stiff-leg derrick boom on shore. Steel tower in rear is on floating concreting plant.

the ground was comparatively dry, and concrete piles were specified. Because of uncertainty as to the refusal depth, the contractor elected to use Raymond concrete piles, whose steel shells could be driven and cut off at the ground surface before the concrete was poured. As against this method, precast piles could have been used, but the contractor felt that it would be difficult economically to order precast piles for the variable and unknown lengths.

Use of Construction Plant and Equipment—Before considering the work on the more difficult of the piers, it is well to visualize the construction plants that were used. Excavation on all of the land piers, for both bridges, was done mostly by crawler cranes operating clamshell buckets. Stiff-leg derrick plants were set up for those piers near the bulkhead line, and floating plants, operating orange-peel buckets, were used to excavate for the channel piers.

Concreting was a mass production job, in which the use of a mobile and flexible plant, consisting of paving mixers, cranes and mast elevators, was practically the unanimous choice. Batch motor-truck delivery, from commercial material yards about a mile from each job, supplied most of the mixers with materials. The exceptions were a batching plant at the dock of the Howland Hook approach, and a central mixing plant, for the

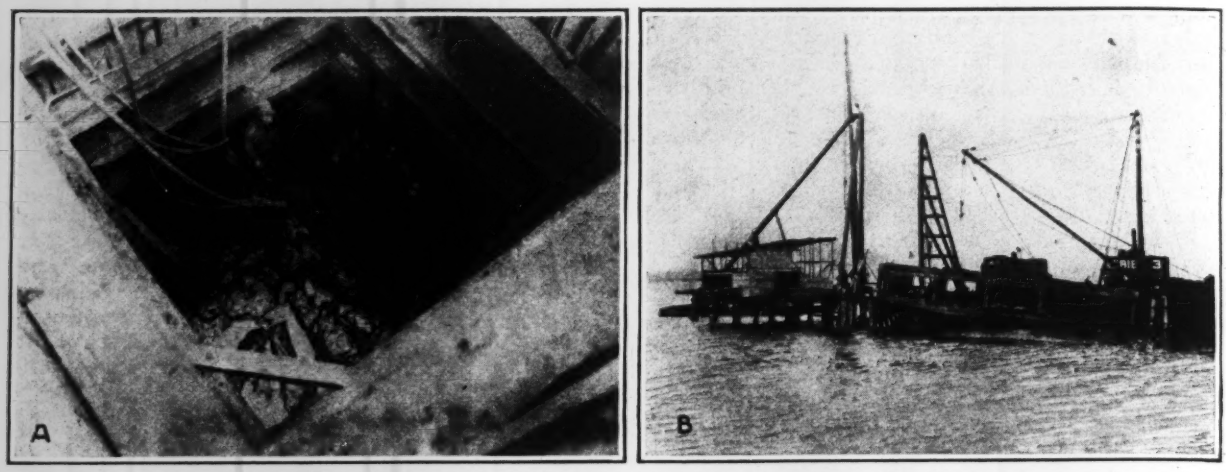

FIG. 4—LAND APPROACH AND RIVER PIER SINKING METHODS

A—Deep pier excavations on Elizabeth approach sheeted to rock with 2-in. wood planking.

B—Distributing the sinking load of steel sheet-piling on cofferdam crib, Outerbridge crossing.

Perth Amboy approach, erected on piles far enough from shore to be reached by material barges at low tide. The latter plant delivered concrete to the piers near the Arthur Kill by means of an industrial railway.

For the bases, the pavers discharged directly. In concreting the plinths on one contract, a support was placed under one end of the paver boom and the discharge bucket was pulled up the incline. For heights greater than this, cranes with 40 to 50-ft. booms, and in one case a 90-ft. boom, were used. With their bottom-dump buckets, they concreted as high as possible, which in some cases was a whole pier, and in others only a part of the shafts. The pavers moved from pier to pier as each was made ready for concreting. When the crane booms became inadequate, mast elevator plants were resorted to. On three of the contracts these were steel mast plants, and on the fourth wooden leads were erected. In some cases, a mast was used for each pier, while in others the mast served two piers by means of counterweight chutes. The maximum tower height necessary was about 160 ft. Electricity, gasoline and steam were used for power, in the latter case a boiler being erected on the dock, and steam piped to the various hoists and pumps. For concreting the river piers, central mixing plants mounted on barges were used.

Channel Pier Construction—Piers *B* and *C* on the Elizabeth bridge and piers, *C, D, E* and *F* on the Outerbridge Crossing can be designated as "channel piers." Both a pneumatic caisson and open cofferdams were used in their construction.

In sinking the pneumatic caisson on pier *C*, Elizabeth bridge, although the theoretically required air pressure was about 31 lb., only 18 lb. air pressure was found necessary, because of a dense clay stratum overlying the rock. The caisson was of wood, 30x77 ft. in plan and some 75 ft. high. It was built at the yard of the Staten Island Shipbuilding Corporation and floated to the site, which already had been dredged as much as possible. In sinking, air spades were used extensively. The use of three material locks, and of three telpher systems of 12-in. I-beams over them, to carry the mud buckets, speeded up the work. Also, by depositing concrete through the three locks, the caisson was easily kept on an even keel during sinking.

The other five channel piers were sunk by open cofferdams. On pier *B*, Elizabeth bridge, a frame of 12x12's was fastened at low water to old dock piles. Ten-foot columns on top of this, and another set of framing completed the preliminary crib, around which 30-ft. 14-in. arch-web steel sheetpiling was driven. As the piling went down, bracing frames were placed at 5-ft. intervals. On this pier, it was necessary to excavate some 20 ft. into the rock, paving breakers removing the top 5 ft. of disintegrated material, while the 15 ft. of solid rock was blasted with 60 per cent dynamite. The steel sheeting was driven to rock and below this 3-in. wood sheeting was used to reduce the flow of water that developed at the toe of the steel piling. One 4-in. and two 6-in. steam centrifugal pumps handled the water which did get in. This pier being located on the sloping river bank, it was necessary to dump about 800 cu.yd. of rock on the stream side of the cofferdam in order to equalize the inward pressures on the bracing.

For piers *C, D, E* and *F* on the Outerbridge Crossing,

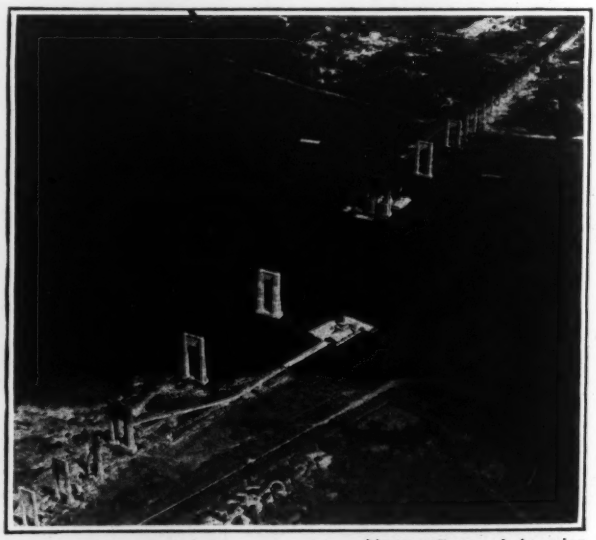

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FIG. 5—SUBSTRUCTURE WORK NEARING COMPLETION ON OUTERBRIDGE CROSSING

Perth Amboy contractor's central concreting plant in the foreground. Tottenville contractor's floating plant anchored at mid-channel pier. Steel erection proceeding on the Tottenville approach.

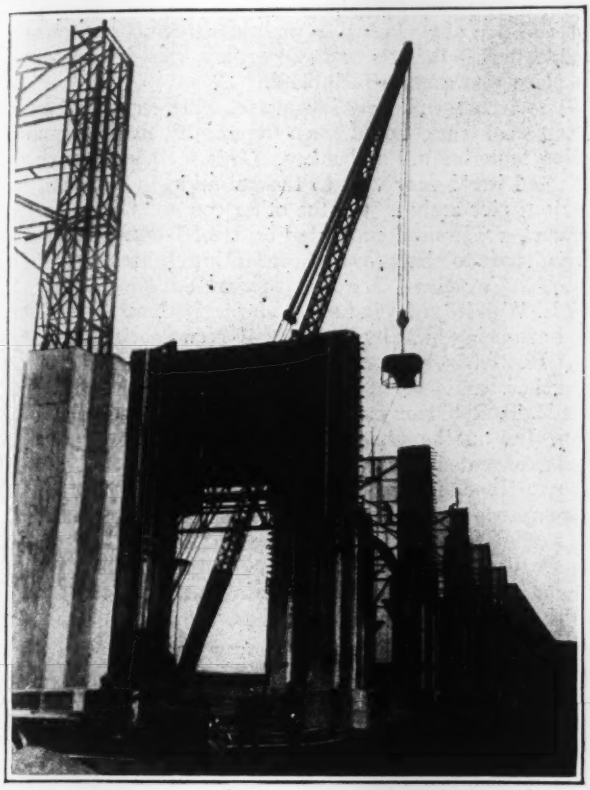


FIG. 6—PLACING CONCRETE WITH 90-FT. CRANE BOOM ON PERTH AMBOY PIERS

wooden cribs were built four tiers high on the shore and floated to place. About two tiers of bracing were added during sinking. Being spotted in the proper location, the cribs were anchored by wood piles driven in several of the pockets, and sunk by weighting with steel piling. Long steel sheetpiling—65 ft. on pier *F* and 75 ft. on piers *C, D* and *E*—was driven, around the cribs and to 20 ft. below the bottom bracing frame. The wood pile foundations were driven with under-water steam hammers, 25 ft. into the sand and gravel. Excavation was done with orange-peel buckets as this type best fitted into the pockets of the crib and between the wood piles.

Concreting was done from barges completely equipped as floating plants. Counterweight chutes, from towers on the barges, placed concrete as high as possible, following which mast plants were erected on the plinths for concreting the top portion of the piers. After the seal course had been placed by tremie, one 10-in., two 8-in., and two 12-in. centrifugal pumps, all direct-connected to single-acting vertical steam engines, removed the water. Thereafter the two 12-in. pumps kept the holes dry.

Land Approach Pier Construction—In constructing the land approach piers, the first necessity was access to the sites. On all of the approaches, this was difficult because of marsh land and lack of roads. The worst conditions occurred on the Howland Hook approach where top soil consisted of 5 or 6 ft. of peat and 5 ft. of grass roots and mud, the whole being in a semi-floating condition. This made it necessary to use platforms for supporting all equipment, and to lay a plank construction road as a preliminary to any actual pier building. By careful operation, a crawler crane operating on mats was able to make a fill from the peaty bog. The fill was covered with sand and a 2-in. plank roadway 20 ft. wide supported on four rows of 12x12-in. sleepers was laid paralleling the bridge approach. On the Perth Amboy contract, too, platforms and plank roads were necessary. One of the reasons why the contractor placed his central mixing plant on piles in Arthur Kill was the difficulty of land transportation of mixing materials and of movement of a paving mixer.

The anchor pier *D*, Elizabeth bridge, required extra heavy construction. Three frames of 12x12's, 5 ft. on centers, were erected on top of the ground, and 15-in. arch-web steel sheetpiling 50 ft. long was driven around them. Braces were added below as excavation progressed to El. —38. Similar methods were used on the first 17 of the approach piers away from the channel, except that straight-web 26-ft. steel piling was adequate. Wood sheeting, 6x12-in. tongued-and-grooved, and 3x12-in. straight lumber, was used on most of the remaining piers, none of which was over 16 ft. deep. On piers 9 and 10, which were on the bank of a small creek, the steel sheeting and wood-pile supports were driven before any excavation was done. Then during excavation braces were placed between the steel sheeting and the pile supports, as well as between the pile supports themselves, to keep the bottom of the steel sheeting from bulging in.

On the Elizabeth approach, where holes 24 to 50 ft. deep were sheeted with 2-in. planking, cross-bracing was used only on the deepest of the holes. The procedure on these piers was to dig only a short distance ahead of the sheeting, tapping it down with small air hammers. Rectangular frames of 10x10's with no cross-bracing were placed on 4-ft. centers. Poling boards were used in some excavations below the vertical sheeting, and in all of the holes air-operated clay spades did excellent service in removing the shale and hardpan overlying the rock.

The unusual soil conditions on the Perth Amboy ap-

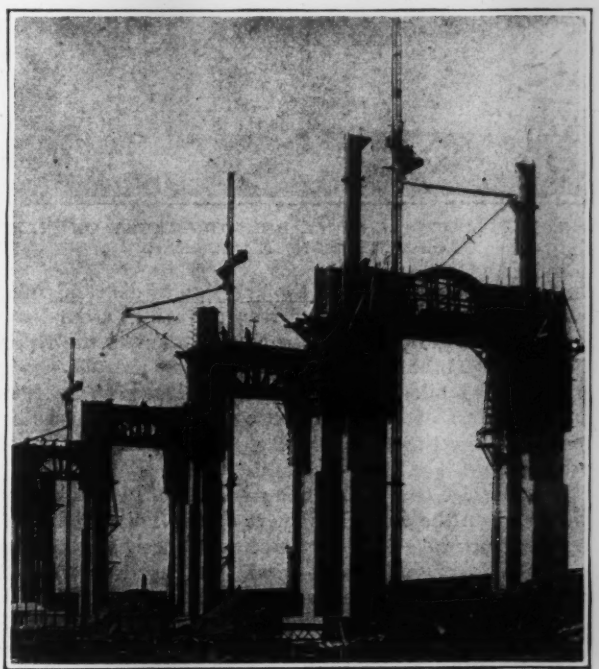


FIG. 7—CONCRETING ELIZABETH PIERS FROM MAST PLANT

On this contract, one mast, by means of counterweight chutes, served two piers, while on other contracts a mast was erected on each pier plinth. On the pier in the foreground, the steel anchorages for the bridge have been placed.

proach, as far back from Arthur Kill as pier 7, required steel-sheeted excavation with extra heavy bracing. On three of the piers, the soil pressure was great enough to cause the lower part of the steel sheeting to come in about 3 ft., and in order to save these holes the seal course of concrete was immediately poured. It was then necessary to drive another row of heavier section steel sheetpiling around the first excavation and to excavate by hand between the two cofferdams. As the excavation progressed, the inner sheetpiling was cut off in about 2-ft. sections and the braces extended to the new sheeting. The successful method finally developed for sinking these holes involved starting the excavation by ordinary methods, driving sheetpiling and placing bracing as the

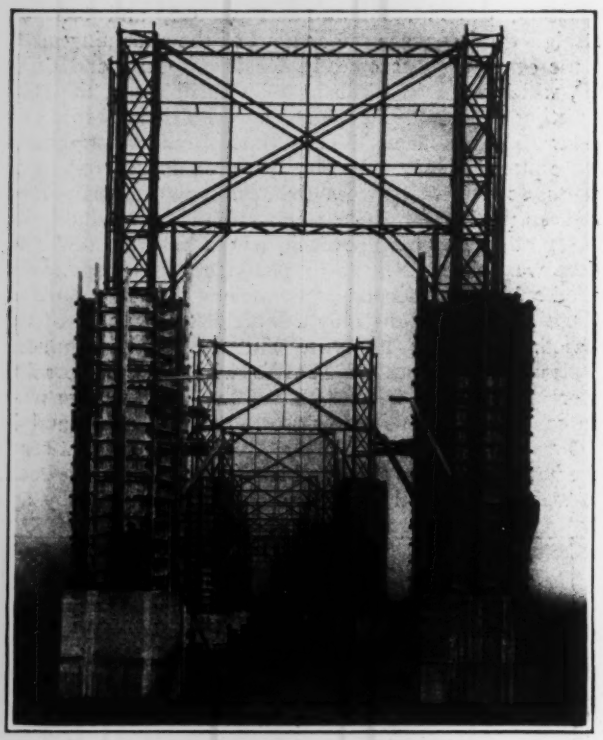


FIG. 8—STRUCTURAL STEEL REINFORCEMENT ON PIERS OF OUTERBRIDGE CROSSING

excavation progressed. At the first sign of distress, three lines of support piles were driven to refusal around the inside of the cofferdam. Then in addition to the regular framing, braces were placed between the pile heads and between piles and the steel sheeting. These piles had the effect of both compacting the soil and assisting the steel sheeting by carrying some of the external pressure.

Structural Steel Reinforcing—On the Perth Amboy piers and on the four river piers of the Tottenville contract, structural steel reinforcement, designed by the contractor's engineer, was used. It has demonstrated a number of advantages, and, although the first cost is larger than for bar reinforcement, the contractor is quite definite in his view that the time saved in erection, in form handling, in concreting, and in earlier completion of the entire work has effected a considerable final saving of profits.

It may be noted from Fig. 8 that the reinforcement when completely erected was a rigid frame. It served to support the formwork for the shaft and for the arch. In addition, by placing a timber, carrying blocks and tackle, across the top of these towers, raising the forms in place was greatly facilitated.

The arch forms on this contract were supported by a small steel truss shored from the plinths, the shores also being built for use as ladders. This method of support adapted itself very well to the variation in radii on the various pier arches. On the other contracts, wood arch centering was used supported on steel I-beams spanning from shaft to shaft, or by wooden kneebraces.

Administration—The bridges were designed by Dr. J. A. L. Waddell, of Waddell & Hardesty, New York City, and are being built by the Port of New York Authority, O. H. Ammann, bridge engineer, and William J. Boucher, engineer of construction. Resident engineers are R. T. Robinson on the Elizabeth bridge, and R. Hoppen, Jr., on the Outerbridge Crossing. The contracts being executed are: Howland Hook, Frederick Snare Corp., Randall Cremer, engineer, and D. H. Cameron, superintendent; Elizabeth, Triest Contracting Corp., C. M. Rauterkus, superintendent; Tottenville, Frederick Snare Corp., John Pierson, superintendent; Perth Amboy, Cornell Contracting Co., S. Kent, engineer, and J. O'Keefe; superintendent.

Use of Returned Sludge Speeds Up Water-Softening Reactions

LABORATORY tests made at Columbus and Pittsburgh indicate that the use of returned sludge hastens water softening reaction considerably, according to A. W. Bull in a paper presented before the Southwestern Water Works Association at Hot Springs in October. During winter months, these reactions are apt to be very slow and often incomplete. A number of water softening plants are adding inert sludge from the coagulation basins to speed up the softening reactions.

The tests carried out at Columbus showed that agitation for 19 hr. was necessary to reduce the soap hardness of water to 66 p.p.m. while the same reduction in hardness was secured in 2 hr. by the addition of 50 c.c. of sludge per gallon of water. Adding 100 c.c. of sludge per gallon of water resulted in the same reduction in hardness in 1 hr. The best results were secured with a concentration of sludge of 15,000 p.p.m. The treatment consisted of 11 grains of soda ash and 11 grains of lime per gallon with a temperature of 17 deg. C. That the results were not due to the presence of unused chemicals is shown by the fact that the final alkalinity and causticity were not increased by increasing the amount of sludge used.

At Pittsburgh, with a water unusually high in magnesium sulphate, it was found that the addition of 7,100 p.p.m. of sludge produced good results. The water was apparently softened as easily at 0 deg. C. as at 10 deg. C. and in both cases better than could have been done at 30 deg. C. without the use of sludge.

Another benefit noted from the use of returned sludge and increasing the concentration of solids in water prior to sedimentation is the clear effluent which is obtained. This is because of the increased area of settling solids, which tends to carry down more of the fine material as it settles.

Investigations in large-scale plant operation should be made before final conclusions are drawn, according to the paper, but the results "give encouraging indications as to what may be secured in practice."

Sand-Gravel as Total Aggregate for Concrete Roads

By W. V. Buck
State Highway Engineer, Topeka, Kan.

Economic concrete road construction in many Mississippi valley states rests on the utilization of local materials varying from traditional standards in size and grading of aggregates. Most Platte and Arkansas river sand-gravels are examples of these off-standard materials excellent in quality in all other respects.

PUMP BARGE AND DISCHARGE PIPE TO SCREEN

THE sand-gravel referred to here is of a relatively fine gradation, ranging from No. 100 material to a maximum of ¾ in. in size. These sands and fine sand-gravels have been used locally for a number of years as total aggregate in concrete bridge and culvert construction with very good results where the proper proportions were used. No concerted effort was made, however, until the past year to determine their suitability for use as a total aggregate in concrete mixtures for pavements. Heretofore, it has been required that a coarse aggregate having a French coefficient of at least 5 should be used with this local material of finer gradation. This meant that it was necessary to ship the coarse material from some outside source, involving the expenditure of a large part of the cost of construction for freight charges on the aggregate alone. The proper proportion of cement to aggregate, using materials of this nature, of course, depends upon the gradation of the aggregate. While this varies considerably with the different localities, the material of the individual pits or beds quite closely follows a certain gradation.

The first paving project on state highway and federal aid work in which specifications permitting the use of sand-gravel as total aggregate were incorporated, was federal aid project 342, section A, in Sedgwick County. Excellent results had been obtained in this county where pit-run material from the Little Arkansas River was used as the total aggregate in the construction of a concrete base for twelve miles of brick road. The success of the material for this purpose suggested the possibility of using it for one-course concrete pavement.

Before writing the specifications for this project, numerous tests, including wear, compression and gradation, were made to determine the suitability and economy of various mixes of sand-gravel aggregate as compared with mixes using coarse and fine aggregate meeting the usual specifications.

The customary mix for concrete pavement construction in Kansas is 1:2:3½, using 1.61 bbl. of cement per cubic yard of concrete. The specifications require that a concrete shall be produced which will withstand an average compressive strength of 2,000 lb. per sq.in. at seven days and 3,000 lb. per sq.in. at 28 days, with standard conditions of manipulation and curing. It is further required that the maximum variation below the average, on individual test cylinders, shall be 10 per cent. The consistency of the mix is determined according to the specifications by the use of the truncated cone, and the maximum slump allowed in paving work is 1 in.

The material for federal aid project 342 was selected from a pit on the bank and in the bed of the Little Arkansas River. It was estimated by test pits that at this location about 4,000 cu.yd. of suitable aggregate were available. Representative samples from these pits, on which sieve analyses were made using the Tyler standard set, showed an average gradation factor of 3.6. This gradation factor can be increased to 4.25 by the elimination of some of the finer material. In order to insure uniform production the specifications provide that pit-run or bank-run aggregate cannot be used but shall be prepared by either screening or water separation so as consistently and uniformly to grade within the following limits :

Retained on a No. 4 sieve not less than 10 per cent nor more than 25 per cent.
Retained on a No. 8 sieve not less than 35 per cent nor more than 60 per cent.
Retained on a No. 28 sieve not less than 75 per cent nor more than 90 per cent.
Weight removed by decantation test not more than 3 per cent.

In consideration of these gradation limits and following the requirement that a concrete having a minimum

KANSAS

IN MANY localities of Kansas, especially in the south, central and western sections, sand and sand-gravel deposits occur in abundance, both in stream beds and in banks or pits. It is a peculiar condition that in these same localities practically no material is available for use as coarse aggregate in concrete mixtures, either in the form of gravel or stone. A method of utilizing these fine aggregate deposits in concrete road construction was an outstanding technical development of 1926.—W. V. Buck, *State Highway Engineer.*

compressive strength of 2,700 lb. per sq.in. at 28 days shall be produced, the specifications state that no less than 1.8 bbl. of cement per cubic yard of concrete shall be used.

Since these preliminary tests have been made, a plant has been installed by the contractor for the production of this aggregate. This plant consists of a sand pump mounted on a barge and the aggregate is pumped to a tipple and there water-separated to meet the grading limits allowed by the specifications. The uniformity of the aggregate produced is demonstrated by the small amount of rejected material. The 500 cu.yd. of material produced and graded shows the following sieve analysis:

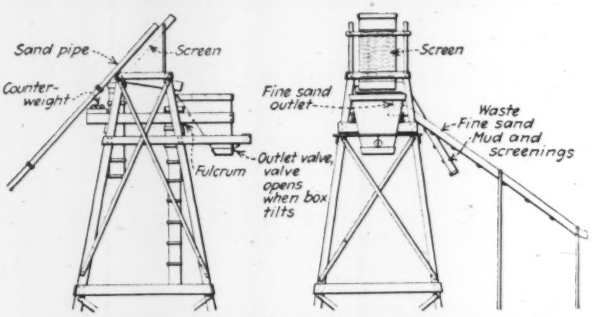
Sieve Size	Per Cent Coarser Than Each Sieve
3	0
½	7.5
4	17.5
8	37.5
14	67.0
28	86.0
48	96.0
100	100.0

The sieve analysis of this material gives a gradation factor of 4.1. The mix designed to meet the conditions

SIDE VIEW OF SCREENING PLANT FOR SAND-GRAVEL

imposed by the specifications, using 1.8 bbl. of cement per cubic yard and producing a concrete of the desired workability and strength, would be 1:3.4 by volume (aggregate dry and rodded).

Due to the fact that fine aggregate mixes will slump proportionately more with the same net amount of water than those where coarse aggregate has been used, it was considered for test purposes that a slump of 1½ in. would be a comparable consistency to the 1-in. maximum slump provided in the specifications. Test cylinders made from mixtures meeting the above conditions with a slump of 1½ in. show a compressive strength of 2,200 lb. per sq.in. at the age of 7 days, corresponding to 3,600 lb. per sq.in. at 28 days. These laboratory tests give strengths higher than required, but it was considered advisable to retain a safety factor on laboratory manipulations until comparable control methods could be established in the field manipulation. In actual practice a slump in concrete of this material of as much as 1½ in. is not necessary for desired workability. Plasticity for proper finishing can be obtained with about ½-in. slump and of course the



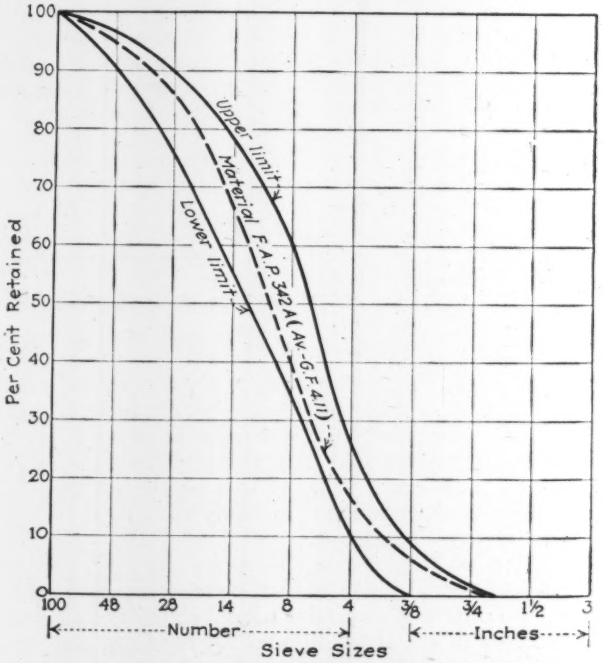
GENERAL PLAN OF SCREENING PLANT SHOWING TILTING HOPPER

resulting pavement will have a relatively higher strength due to the decreased amount of water used.

The economy effected in the development of this material is reflected in the contract prices for work awarded under these specifications. On account of the ease and low cost of production of the aggregate, even though a slightly greater quantity of cement is required, the cost of concrete pavement has been reduced 15 to 20 per cent in the sections of the state where good coarse aggregate is not available. A part of this reduction can also be attributed to the fact that only one aggregate need be handled at the proportioning plant and to the greater ease of finishing the pavement.

The equipment necessary to produce and grade the aggregate is simple and inexpensive. The material is pumped over a screen mounted on a tipple. This screen rejects the mud balls and other undesirable coarse material and passes the remainder, including water, to a V-shaped bin, that is mounted on a beam counterbalanced by a certain weight.

The separation of the excess fine material is effected by a flotation process and this material together with the water passes out through an overflow from the bin. The sizes rejected are smaller than would be retained on a No. 28 standard sieve.and the grading can be con-



RELATION OF SPECIFIED LIMITS OF GRADATION TO AGGREGATE USED

trolled by the weight of the counterbalance used. The heavier the weight, the longer the material is retained in the bin before it tilts, opening the gate at the bottom and discharging on to the stockpile. During this time the water and fine material are being removed through the overflow.

In experimenting to obtain the proper gradation it was found possible to produce an aggregate coarser than the specified upper limits by increasing the counterweight, and that this controlling device was quite flexible in obtaining a material having a desired gradation factor.

While a large quantity of aggregate has been produced during the winter months, no pavement was laid during the past season. It is not possible, therefore, to report field observations and tests. Laboratory tests and observations of city pavements show that there is every reason to believe that just as satisfactory a wearing surface will be provided using sand-gravel as total aggregate, as with fine and coarse aggregate, and that an equal or better riding surface will be produced. The production of a smooth riding surface is more easily obtained than with coarse aggregate, due to the workability of the concrete. Any irregularities shown by the straight-edge can be corrected without removal of as much material as is necessary with coarse aggregate concrete and the initial work of the finishing machine is smoother.

The first section of road to be constructed under these requirements will consist of 5.2 miles of concrete pavement of the Bates type having a cross-section of 9-6-9. Plans have been prepared for several additional miles.

Development and use of these local aggregates to produce a satisfactory pavement mean a large saving in construction costs to those sections of the state in which these deposits occur and will largely increase the mileage of paved roads to be built. The Shultz Construction Co. of Wichita, Kansas, has the contract for the construction of federal aid project 342, and the resident engineer for the Kansas highway commission is L. K. White, Wichita, Kan.

Conditions at Sewage-Works of Milwaukee, Wis.

By Robert Cramer
Chief Engineer Sewerage Commission,
Milwaukee, Wis.

Abstract of a paper read before the Lake Michigan Sanitation Congress.

TREATMENT of the sewage at Milwaukee is at least as good as that obtained in any sewage disposal plant of which I have knowledge. It is far better than that obtained in all but very few. The purification possibilities of the activated-sludge process are being demonstrated on a large scale. We are treating an average of 70 m.g.d., as against 55 m.g.d. four months ago, the former being 83 per cent of the amount which it was estimated the district would yield at this time. With all sewage from the district turned in, the total will be 95 to 100 m.g.d. We are thus taking in about 75 per cent of the total amount [dry-weather flow] now produced, and we expect to turn in all of it in about three months, when additional machinery is installed.

The reason why the total dry-weather flow is not treated is found in the difficulties encountered in operating that part of the plant which dewaters the excess sludge and converts it into a commercial fertilizer. These difficulties were due in part to faulty design resulting in insufficient capacity of some apparatus, and in part to lack of experience and proper organization, resulting in incompetent handling of some apparatus. We have now modified some of our apparatus and have arranged for more efficient operation. Final disposal of the sewage solids by sale of the dried sludge in the fertilizer market seems assured; the demand for it is insistent and we have our output sold for six months in advance.

Development of Processes—Among the changes and additions are some of outstanding importance, particularly the rebuilding of the drier settings so that the excess air necessary for drying is now mixed with the gases of combustion in the furnace rather than after the gases leave the furnace. We thus maintain low furnace temperature and can make the brickwork last longer. The air ports of the drier drums have been so changed that none of the dried material can spill into the furnace. Further, we have learned to keep the temperature in the driers so steady that the pyrometer charts appear as though they had been drawn with a ruler.

Although one of the driers has not been changed, we have maintained an output of over 90 tons of fertilizer per day and an average output of 70 tons per day for nearly two months. It seems that the daily output is practically limited only by the moisture content in the filter cake which is put through the driers, and the control of this moisture is one of the difficulties. To meet the varying seasonal and weather conditions, we must be prepared to use upon short notice different chemicals for treating the sludge before it goes to the filter. At different times we use sulphuric acid, alum or ferrous sulphate-ferric chloride, and sometimes a combination of these.

Pumps provided for moving the sludge to the filters proved inadequate for two reasons: (1) The material which they handled is heavily charged with air and the volumetric efficiency of the pump is therefore low; (2) to meet variations in the character of the sludge it was necessary at times to run with a solid content as low as 1 per cent, while the pumps were proportioned for 2 per cent. Additional pumping equipment now being installed will enable us to handle the full quantity of sludge expected under maximum conditions. As the chemical condition of the sludge had a very corrosive effect on the iron parts, these pumps are being rebuilt with bronze parts.

By-products—For four by-products no definite final disposal has been found. (1) The coarse screenings do not make a very large quantity, so that this problem will not be pressing for some time, and we hope to be able to handle the coarse with the fine screenings. (2) The grit from the grit chambers also is not a very large quantity. We have spread some on our grounds and there has been some demand for filling. (3) The fine screenings present a problem. It has been suggested that they be pressed and then incinerated, but presses installed proved impractical, the handling of the wet screenings as well as the pressed cake being very difficult. Small-scale experiments have been made with a high pressure cooker, which produces a salable fertilizer, and with biological digestion, which gives an inert end product that may either be dried and spread on the land or mixed wet with the sludge from which fertilizer is made. Both methods gave promising results and we are preparing to try them on a sizable scale. (4) As customers object to a high percentage of dust in the fertilizer we separate it and have tried mixing it with the wet fertilizer cake before the cake enters the driers; but the dust reappears and must be removed finally. Experiments indicate that if this dust is discharged into the sewage and sent again through the plant it will incorporate itself in the larger particles.

Odor from our drier discharges has been a troublesome problem. Under certain wind conditions I have noticed it six miles from the plant. Washing the drier gases with a water spray proved ineffective. We now treat them with chlorine in gas form and although the odor of chlorine is often quite noticeable, especially near the plant, we now have very few complaints.

The testing and control station is now in full daily operation on a program of investigation which has been laid out by our chemists for several months in advance. This control station consists of four small plants, each complete and independent in itself, and having its own return sludge circuit.

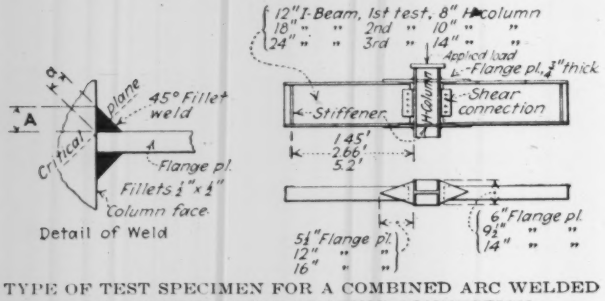
In cost of operation, we have somewhat reduced the labor item and the coal consumption, while indications point to a possible considerable reduction of the operating cost.

Wind Bracing Connection Combines Welding and Riveting

By W. A. Hakin
Structural Engineer, Albert Kahn, Inc., Architects, Detroit

and R. G. Richards
Sales Manager, Slaymaker Electric Welding Co., Detroit

TESTS were recently completed by the Detroit Structural Welding Committee on a combined arc-welded and riveted wind bracing connection between floorbeam and column. The type of double cantilever specimens used is illustrated in the drawing. In designing the joint it was planned that the riveted connection should carry the vertical shear and that the flange stress should be transferred by welded flange plates. The sole function of the weld, therefore, was to act as an end restraining medium subject to induced tensile and horizontal shear forces only.



TYPE OF TEST SPECIMEN FOR A COMBINED ARC WELDED AND RIVETED WIND BRACING CONNECTION

The Detroit welding committee was formed by structural engineers, fabricators, welding companies and others in Detroit interested in the possibilities of welded structural design.

Tests were made using 24-in., 18-in., and 12-in. I-beams. The majority of the failures occurred through the body of the weld, the angle of the critical plane being about 45 deg. The results of the tests were tabulated as shown in the accompanying table for the tests on the 24-in. beams.

For computing the unit value of the weld bead the gross area rather than the net area along the critical plane was used. "Percentage of effective weld area," shown in the table, was obtained by dividing the net area of the fracture by the gross area, both areas being taken along the critical plane. A comparison of the strength of the net section with that of the gross section yielded high values for welds of poor quality, which was the primary reason for the adoption of the gross area rather than net area as basis of computation of unit strength.

For the 24-in. beams it will be noted that the ultimate strength per 1-in. run of ½-in. welds varies from a minimum of 5,500 lb. to a maximum of 12,250 lb. For the 18-in. I-beams, the range was from 16,400 lb. to 11,675 lb. and for the 12-in. I-beams from 8,400 lb. to 11,200 lb. For the 18-in. I-beams the ultimate strength per square inch of contact varied from a minimum of 16,400 lb. to a maximum of 22,900 lb. and for the 12-in. I-beams the range per square inch was from 16,800 lb. to 22,400 lb.

Tracklaying Practice on the Rock Island Lines

Organization for Materials Train, Front Gang, Back Gang and Ballasting—Special Devices Expedite Work

ACTUAL conditions and suggested variations therefrom in the methods of laying track on the new Amarillo-Liberal line of the Chicago, Rock Island & Pacific Ry. are given in this article, which is based on information from F. T. Beckett, engineer of maintenance-of-way, who was in charge of this part of the construction work, the tracklaying being done by railway forces. This detailed information supplements that in the article on this new line in *Engineering News-Record* of Aug. 18, 1927, p. 252.

Materials Train—This train was made up as follows: Steam ditcher on head end (used as pioneer car to handle rails and ties); 2 cars of rails; locomotive; 1 water car; 1 car of angle bars; 1 car of spikes, bolts and nutlocks; 1 car of tie-plates; 5 cars of ties, averaging 270 ties each.

The 100-lb. re-rolled rails received from the mills (and weighing about 90 lb. per yard) averaged about 110 rails per car. If it is possible to hold rails on the cars, it is more economical to do so and to lay them from the cars as received from the mills, dropping off and picking up later any short-length rails which it is not wished to lay. If the rails are first unloaded and piled, they should be reloaded, with 140 to 150 rails to each car, turning half of them one way, the other half opposite, so as to avoid turning rails when laying. This is to comply with instructions requiring a certain side of the rail as the gage

TEST RESULTS FOR COMBINED ARC WELDED AND RIVETED WIND BRACING CONNECTION OF 24-IN. I-BEAMS (79.9 LB.) TO 14-IN. H-COLUMNS

Test No.	Ultimate Applied Load, Lb.	Support Reaction, Lb.	Effective Lever Arm, Ft.	Ultimate Bending Moment, Ft.-Lb.*	Flange Stress, Lb.	Approx. Length of 1-In. Bead	Ultimate Strength per 1-in. Run of Weld, Lb.	Deflection — At Elastic Limit, In.	Deflection — At Ult. Load, In.	Approx. Contact Area "A" at Column Face, Sq.In.	Net Area "a" of Fracture, Sq.In.	Percentage of Effective Weld Area	Unit Weld Strength — On Gross Area, Lb. per Sq.In.	Unit Weld Strength — On Net Area, Lb. per Sq.In.	Remarks
1	187,450	93,725	5.2	487,000	243,500	29	8,400	0.402	0.59	14.5	8.8	85.5	16,800	19,600	Weld failure at column
2	224,000	112,000	5.2	582,000	291,000	29	10,000	0.326	0.625	14.5	9.8	95.5	20,000	21,000	Weld failure at column
3	273,120	136,560	5.2	710,000	355,000	29	12,250	0.366	0.75	14.5	....	....	24,500	.....	No failure
4	259,900	129,950	5.2	675,000	337,500	29	11,625	0.329	0.68	14.5	....	....	23,250	.....	Fracture of flange plate
5	250,000	125,000	5.2	650,000	325,000	29	11,200	0.265	0.375	14.5	....	....	22,400	.....	No failure
6	273,900	136,950	5.2	712,000	356,000	29	12,250	0.399	0.84	14.5	....	....	24,500	.....	No failure
7	268,300	134,150	5.2	696,000	348,000	29	12,000	0.46	1.06	14.5	....	....	24,000	.....	Fracture of flange plate
8	122,000	61,000	5.2	318,000	159,000	29	5,500	.....	0.81	14.5	4.78	46.6	11,000	23,500	Weld failure at column
9	184,430	92,215	5.2	480,000	240,000	29	8,275	0.39	0.56	14.5	4.9	47.4	16,550	34,500	Weld failure at column

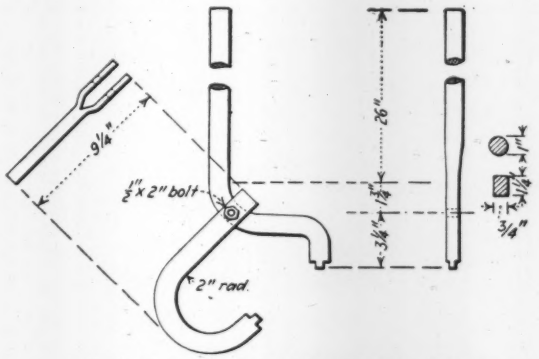
*Capacity of 24-In. I at 79.9 lb. = 435,000 ft.-lb. at fibre stress 30,000 lb./sq.in.

side. In laying rail in some of the deep, narrow cuts, it was found necessary to unload and reload in this manner, as rails could not be turned in these cuts.

Ties are kept in the cars until needed, and are unloaded by the front or materials train from cars to grade, just back of rail laying. They should be unloaded on a fill about 4 ft. high if possible. Then they are reloaded onto trucks which haul them ahead and distribute them on the completed grade ahead of rail laying.

Men on Materials Train—There were three men on the rail car to mark with chalk the center of rail, where clamps were to be attached; to turn the rails and get them in work position, to apply rail clamps, and to guide and steady them when picked up by the crane. The number of men required to handle ties will vary, but on an average this work will require 20 men in charge of an assistant foreman. Of these, 12 will be unloading ties from car to ground and 8 placing ties on trucks. This is based on light Mexican laborers, who lift only very light loads.

For the track fastenings, a 1-ton truck and 3 laborers will unload from cars, truck ahead and place on ground sufficient fastenings such as spikes, bolts, nut-locks and tie-plates. All angle bars, bolts and nut-locks are trucked but only sufficient spikes and tie-plates to plate and spike



ANGLE-BAR TONGS FOR TRACKLAYING

every fourth tie. The remaining spikes and tie-plates are unloaded along the track from the train as required.

Organization Ahead of Train—For distributing and spacing ties, there is an assistant foreman with 10 men who properly space and line the ties ready for rail to be laid. For laying rail: 1 foreman, 1 timekeeper, 1 assistant foreman, 3 rail men, who handle rail as swung by crane, turn it if necessary and place it in proper position on the ties; 8 spikers; 2 bolt men; 2 applying angle bars as soon as rail placed; 1 placing small material, 1 with angle-bar clamp.

The angle bars are applied to the rail when laid by putting in and partly tightening one bolt. The rail is then slipped in between the angle bars, and a clamp or angle-bar tong is applied to force the bars into position against the rails, ready for bolting. This device, which is commonly used also in rail renewal work on the Rock Island Lines, is shown by the accompanying drawing.

To fasten the joint quickly and temporarily, a key-bolt is used, consisting of a ⅞- or 1-in. bolt 6 or 7 in. long, having a slot or keyway ½ in. long cut through the threaded portion and about ½ in. from one end. A flat taper key or wedge is made to fit this slot. With the bolt slipped through the splice bars and rail, the key is dropped into the slot, holding the joint firmly until the materials train has passed over. When the back gang completes the bolting, the key-bolt is removed as soon as one regular track bolt has been inserted at each end of the joint and the nuts are screwed up. It is then replaced with a standard track bolt, the key-bolt being carried ahead to the rail-laying gang. This device, which originated with the foreman in charge of tracklaying, was found to be a great time saver, as by the time four ties are spiked the joint is secure and train may move ahead one rail length. This rail-laying gang can average 4,000 ft. of track daily.

Organization Behind the Train—For the full bolting, full spacing, tie-plating, lining and surfacing, the following gang can do all back work and complete 4,000 ft. of track daily. If the subgrade is uniform, it is not necessary to do any surfacing previous to applying ballast, except at occasional spots, and these can be leveled up by the lining gang.

1 Foreman
2 Assistant Foremen
1 Timekeeper
6 Men with push car to distribute tie-plates, spikes, bolts and nut-locks
10 Sets of spikers (20 men)
10 Nippers
6 Boltmen
8 Lining
4 Men with push car to clean up, pick up any surplus fastenings and move them ahead.

Track Ahead of Train—The track has the full number of ties placed ahead of the train, if it is possible to do so. In some short stretches it could not be done on account of rains making slow work for trucks, and only about half the number of ties could be placed ahead of rail laying on the grade. In such case, the additional ties required are unloaded back of rail laying and are applied by the gang doing back work.

Only one track bolt is applied to each joint ahead of train. The joint is fastened temporarily by means of a special slotted bolt with wedge key which is removed after the train passes and replaced with a standard track bolt, as noted above. Only every fourth tie is spiked ahead of train, and with the heavy rails this was sufficient. Not a single derailment to front train occurred due to rails spreading on account of insufficient spikes.

Ballasting—The ballast should be kept up as close to the completed back work as possible, especially the unloading, as it strengthens the track and saves any surfacing being done ahead of distributing ballast. The gang doing the inserting should follow the unloading as closely as possible. With sufficient ballast, the inserting gang should be about two miles back of the finished rail laying. This gang should be large enough to complete 2,500 ft. per day on the first raise. To do this and also handle the unloading will require about 75 men. This arrangement is made because there is not at all times completed grade ahead for continuous rail laying, and the front gang will at these times drop back and assist with ballast work. An additional gang on ballast would be required if the rail gang continued laying daily.

The makeup of ballast train will depend on the kind of ballast and length of haul. If the ballast is obtained locally along the line, as was the case in using the caliche on this line, and if the haul is short, then there should be at all times good ballast cars available, 15 cars per train with a short haul not to exceed 10 miles. Caliche ballast, as well as the pit-run gravel from Sand Springs which contains many large rocks, is hard to unload and an average of 12 men under an assistant foreman was required to unload it from cars.

Concrete Arch Bridge over the Mississippi

Design and Construction of Bridge on Twin-City Highway—Arch Ribs with Spandrel Bents—Foundation Work—Cableway for Concreting—Steel Centering

By M. S. Grytbak
Bridge Engineer, Department of Public Works,
St. Paul, Minn.

A NOTEWORTHY addition to the many large concrete highway bridges in the St. Paul-Minneapolis region, the Intercity Bridge across the Mississippi River, was completed in June. Popularly it is called the Ford bridge from its proximity to the motor plant at the east approach, and it connects Ford Ave., St. Paul, with Minnehaha Park on the west side. Beyond this, a fill and paved approach 1,000 ft. long, including a three-span bridge over Godfrey Road, extends to the street system of Minneapolis. The bridge is a short distance above the U. S. Government dam and power station, this site being selected for the advantage of close access to the motor plant, while it also provides convenient separation of grades between the bridge and the boulevards on both sides of the river. A general view of the structure is given in Fig. 1.

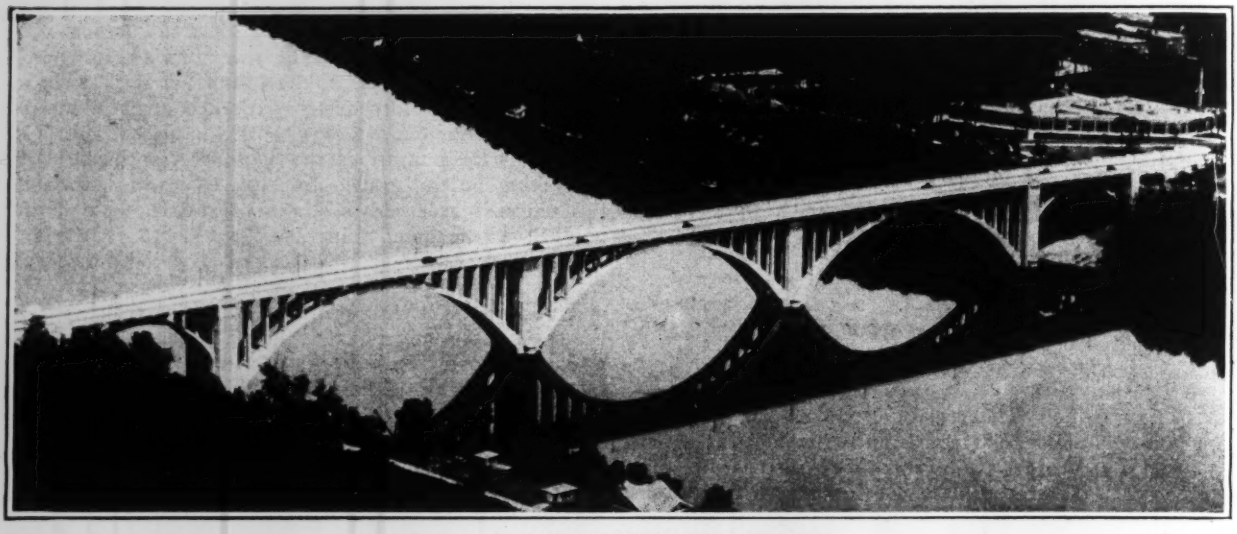


FIG. 1—ST. PAUL-MINNEAPOLIS BRIDGE

In this vicinity the river has a narrow gorge nearly 170 ft. deep in the loose sandstone rock, with 35 ft. of sand, gravel and boulders on the bottom. The depth of water below the crest of the dam, at El. 743.5 Cairo datum, averages 35 ft., but the bottom has 10 to 12 ft. of soft sludge deposited by the city sewers discharging above the dam during the past fifteen years.

Bridge and Approaches—The total length of the bridge between faces of end pedestals is 1,521.5 ft., with three main arches of 300-ft. clear span, flanked on each side by an arch of 139-ft. clear span. The west arch reaches the foot of the steep bluff, requiring only a short girder span to reach the bank. On the east side, however, the line of the bridge is nearly identical with a ravine which required four concrete girder spans and considerable filling to reach the bank. A concrete viaduct of four 40-ft. arch spans, at right angles to the bridge, carries the river-front boulevard under the bridge approach.

Piers 1 to 5 of the east approach are three-post bents with the outer columns connected by heavy struts a few feet below the finished ground level and supporting curtain walls between the columns, so that with the ravine filled the approach has the appearance of retaining-wall construction. The transverse opening at pier 5 is closed by the floorbeam, which is made sufficiently deep for this purpose. The underside of the floorbeam at pier 6 has the same elliptical profile as the floorbeams over the arch spans, the opening from the undercrossing being protected by a railing. The approach spans are of the ordinary girder type, 23 ft. 9 in. to 43 ft. 9 in. between supports.

The bridge roadway is 40 ft. wide, with two 9-ft. sidewalks. The boulevard roadway is 30 ft. wide, with two 6-ft. sidewalks. From water level to bridge floor the height is approximately 100 ft. Grades of 2.54 per cent from the east end to pier 6, and 1 per cent from the west end to pier 11, are connected by a vertical curve with a slight grade change at every panel point.

Arch Spans—Each of the three 300-ft. spans has two five-centered arch ribs with a rise of 87 ft. $4\frac{1}{8}$ in. above the springing line. Their cross-section is 5x12 ft. at the crown and 9 ft. 9 in. x 12 ft. at the springing. Each of the two 139-ft. spans has two similar arch ribs with a rise of 48 ft. $11\frac{1}{2}$ in. and cross-sections of 2 ft. 9 in. x 12 ft. at the crown and 4 ft. $10\frac{5}{8}$ in. x 12 ft. at the springing. In all these spans, the ribs are 32 ft. c. to c. The floor panels over the 300-ft. spans are 18 ft. 2 in. c. to c. of floorbeams and spandrel columns, while those

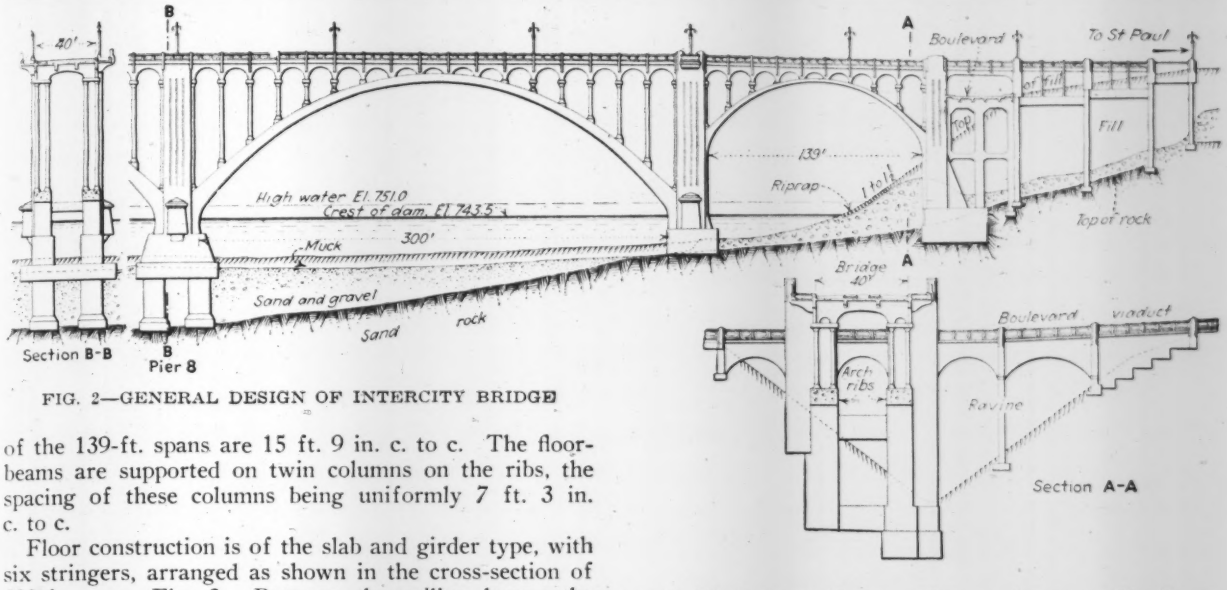


FIG. 2—GENERAL DESIGN OF INTERCITY BRIDGE

of the 139-ft. spans are 15 ft. 9 in. c. to c. The floorbeams are supported on twin columns on the ribs, the spacing of these columns being uniformly 7 ft. 3 in. c. to c.

Floor construction is of the slab and girder type, with six stringers, arranged as shown in the cross-section of 300-ft. span, Fig. 3. Between the railing beams the surface of the slab is horizontal, the 4-in. crown in the pavement being made up by a 1 : 4 : 7 concrete fill. Upon this fill is a $\frac{3}{4}$-in. sand cushion for the 3-in. brick pavement. On the floor slab is a two-ply cotton-fabric waterproofing membrane, protected by a 2-in. coat of 1 : $2\frac{1}{2}$ mortar.

Expansion joints $\frac{1}{2}$ in. wide over the arch spans are provided at every third panel. The sliding of the stringers is taken care of by a $\frac{1}{2}$-in. rolled bronze plate between two $\frac{1}{2}$-in. steel plates, the bronze plate being riveted to the lower steel plate and both steel plates being anchored to the concrete. By this construction it is hoped to eliminate corrosion of the plates, and to make the joints as effective and non-resisting as possible. Fig. 4 shows the expansion joints and plates used over the 300-ft. spans. All expansion joints in the slab underneath the

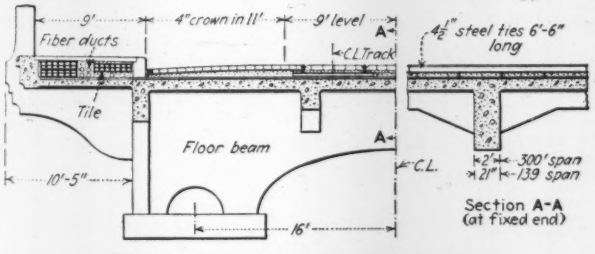


FIG. 3—CROSS-SECTION OF FLOOR

waterproofing are covered with soldered strips of copper bent down in a V-groove at the joints and anchored in the concrete at the edges. The V-grooves are filled with a plastic asphalt composition. The floor of the viaduct undercrossing is waterproofed in a similar manner but here the floor slab itself is crowned and the 3-in. sheet asphalt paving is laid directly on top of the 2-in. mortar protection.

Miscellaneous Features—All piers, except a few of the smaller approach piers, are on the solid sand rock, the two middle river piers, 8 and 9, being each supported on four cylindrical concrete caissons carried to rock about 70 ft. below water level. In the larger piers, the upper body is hollow with 2-ft. walls, and was constructed as two units, with the open faces toward each other. The two half piers are tied together at the water line by a heavy wall and at the top by the floorbeams. Half-columns corresponding to the spandrel columns face the sides of the piers.

The curbs are 12 in. high above the pavement and the space under the sidewalk is filled with conduits and hollow building tile laid in cement mortar. Railings were poured in place, with satisfactory results. They are of 1 : 2 : 3 concrete, made with gravel for coarse aggregate, and are heavily reinforced. A rubbed finish was given to the railings and the bridge approaches, including the underside of the bridge floor at the undercrossing. The remainder of the structure received no special surface treatment. Reinforced concrete buried in the ground is given two coats of asphaltic paint and all pockets in the piers where water may collect are waterproofed and drained. Special care was taken to eliminate or fill with concrete all places where water and ice may collect, and cast-iron scuppers and drain pipes are provided at the curbs of the bridge and undercrossing.

Careful consideration was given to the accessibility of all points of the bridge for inspection or repair. Open spaces between the arch columns, between the top of the ribs at the crown and the floor, and through the pier walls at the springing of the arches, make it possible to walk over the ribs for the entire length of the bridge. By similar holes through the pier walls and floorbeams at the tops of the column capitals, the floorbeams and stringers may be reached at any point. Galvanized steel $\frac{7}{8}$-in. rungs are concreted in for the purpose of reaching the inside of the piers. These features were found convenient during the construction work.

Reinforcing steel is of the intermediate grade, with $1\frac{1}{4}$-in. round bars for the ribs. Bids were also received on the plan of using five structural steel ribs in each of the 300-ft. arch ribs, but the reinforcing bars were found to be considerably cheaper. Longitudinal rod splices in the ribs have four $\frac{1}{2}$-in. U-clamps, with a $\frac{1}{2}$-in. plate washer, arranged in groups of two at each end of the 4-ft. splices and spaced 6 in. c. to c. Between the long rods, $1\frac{1}{4}$-in. square bars 6 in. long were placed for the purpose of spreading the rods and improving the bond. All reinforcing steel was detailed complete before being submitted to the steel company for manufacture. Com-

bination light and trolley poles are spaced approximately 110 ft. c. to c. on the curbs. Lights for the undercrossing and navigation lights at the channel span are also provided.

Construction Work—Construction was commenced in August, 1925. The first concrete was poured Aug. 25, 1925, and the last concrete in the bridge floor on Nov. 19, 1926, or less than fifteen months later. The bridge was opened for temporary foot traffic during the winter and the remaining paving, sidewalks and railing were completed in June, 1927. For the foundation work during

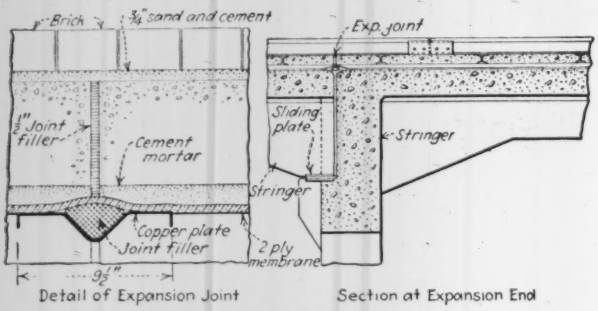


FIG. 4—EXPANSION JOINTS

the winter of 1925-26, the concrete materials were heated and the placed concrete was protected by covering and artificial heat.

The principal equipment was a 15-ton cableway of 1,900-ft. span, with movable timber towers 135 ft. high. The concreting plant, located on the west bluff, had two 1-yd. mixers, equipment for measuring aggregates by weight, a cement bin and belt conveyor for feeding the bulk cement into a suspended steel hopper over the mixers, and a 120-ft. belt conveyor inclined 25 deg. for transporting the mixed concrete to the floor of the bridge. Two 2½-yd. cable buckets then carried the concrete into place. The cableway, mixers, pumps and most of the derrick hoists were operated electrically.

Borings—Wash and core drill borings showed an average depth to sand-rock of approximately 70 ft. below water level near piers 8 and 9, at the middle of the river. At piers 7 and 10, the depth varied from 20 to 35 ft., and in the ravine, at pier 6, it was approximately 20 ft. below the dam crest, rising steeply between bents 4 and 5. The upper stratum of sand-rock found at piers 8 and 9 was of gray color and considerably harder than the yellow or white St. Peter sand-rock above and below this level. Cores were obtained in the gray sand-rock, while it was impossible to obtain them in the yellow rock. The latter, below the level of the gray rock, was also water-bearing, with sufficient pressure to force the water in the pipe several feet above the surface of the river.

In a few cases the results found by the borings did not agree with the results found by the excavations. One reason was that even the broken or pulverized sand-rock would pack together under the overburden tight enough to give large resistance to the drill pipe. Also, the porous but solid sand-rock under water pressure will crumble so that the water at some point may rush in and fill the cofferdam in a few minutes. During the construction at piers 7 and 10 this would invariably take place where the sheeting was driven into the sand-rock and the water pumped out. These conditions required several of the cofferdams to be sealed under water, and pier 10 had to be extended deeper than was expected.

Foundations—The caissons for piers 8 and 9 are 40 to 43 ft. long and extend about 30 ft. below the concrete seal of the cofferdam. The inside shaft diameter is 10 ft. and the concrete shell 2 ft. thick, but increased to 3 ft. for the top 6 ft. of the shell, in order to give a 1-ft. projection on the outside of the caissons. This projection embedded in the 10-ft. cofferdam seal anchored the seal against the uplift when the cofferdam was pumped out. Fig. 5 shows the caissons and forms.

The sinking of these caissons and the construction of the sheetpile cofferdams in deep water constituted the most difficult part of the work. Cofferdams were built of steel sheetpiling of ½-in. web thickness driven into the gravel bed with a penetration of 8 to 12 ft., and 12x12-in. timbers for temporary bracing were forced down inside the cofferdams. This bracing was framed with openings of sufficient size to permit lowering the cutting edges of the caissons, 19 ft. in diameter. It was found impossible to unwater the unsealed cofferdams and the contractor therefore found it necessary to use tight steel forms for constructing the caissons. These forms were made up in sections 5 ft. high, of ¼-in. plates and with 2½x2½x¼-in. angles at the ring joints.

Construction and sinking of the caissons proceeded as follows: The floating form for the cutting edge was filled with concrete to the bottom of the vertical reinforcing steel, thereby sinking the form to within a few feet from the top. Additional inside and outside sheets were then bolted to the bottom form, which was held in control by cables. The balance of the reinforcing steel, consisting mostly of vertical bars which had been framed on shore, was then taken out bodily by the cableway and inserted into the form. The ropes were slacked off until the form rested on the sand bottom, when it was filled with concrete through the water by means of tremies until the buoyancy of the pumped-out form was overcome. The form was then emptied of water and the concreting of the form completed in the dry.

After the concrete caissons had hardened for a few

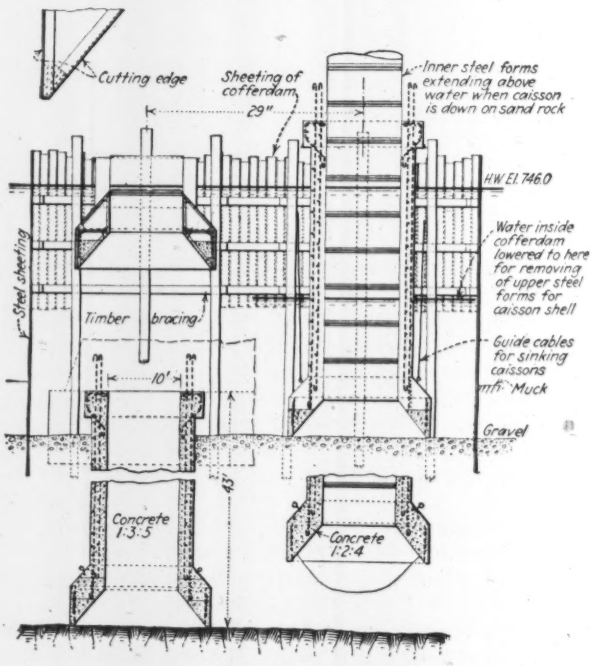


FIG. 5—DESIGN OF PIERS

FIG. 6—STEEL CENTERING FOR 300-FT. ARCH

days the sinking to rock was begun. This was accomplished by means of orangepeel and clamshell buckets which were lowered to the bottom through the water inside of the caissons. As the material was removed and the sinking proceeded, additional inside form rings were added, forming a guide for the buckets through the water. The sinking proceeded rapidly, at an average of about 4 ft. per 24-hr.-day, the cofferdam bracing serving as guides for the caissons. Most of the material removed was deposited inside the cofferdam, replacing material which would scour in along the outside shell of the caisson.

With the cutting edge within 3 or 4 ft. of the sand-rock, it became necessary to use a high-pressure water jet to clean out the sand and gravel under the bridge. No attempts were made to unwater the inside of the caissons, as the thin steel form extending from the top of the concrete to the water surface could not withstand the water pressure, nor was the connection between the cutting edge and the sand-rock tight enough to stand the 70-ft. head of water without a blow-in. The inside of each caisson was therefore concreted through the water after the sand-rock had been inspected by a diver. The preliminary timber bracing of the cofferdam was next removed, and after excavating to the required depth, the 10-ft. concrete seal was poured through the water by means of a 2-yd. marine dump-bucket having a cover which opened automatically as the bucket was raised. A new set of bracing was forced down inside the cofferdam, so that it could be pumped out and the pier forms started. The total movement of the sheeting did not exceed the 2-ft. allowance at pier 9, but at pier 8 part of one side of the cofferdam moved sufficiently to require its redriving.

During the construction of pier 10 it was found necessary to extend the excavation about 12 ft. deeper than planned. While the borings had shown firm yellow rock, the bottom was found to consist only of compact but pulverized white sand particles which would be unsatisfactory for the foundation. In order to save the cofferdam already driven, with 30-ft. steel sheeting, it was decided to drive steel piling inside the pockets formed by the bracing and carry the excavation down to firm rock. This plan was carried out successfully. Eight small individual piers were put down and the cover slab was reinforced to distribute the load over these piers.

Arch Centering—All arch ribs were built on steel centering supported on timber towers, as shown in Figs. 6 and 7. A view of this centering was given in *Engineering News-Record* of Sept. 2, 1926, p. 371. The towers were made of double 14x14-in. posts under the center span trusses, heavily braced, and the whole carried on groups of piling driven to refusal into the river bottom. On account of the great cost of lowering the water these piles were braced only to about 4 ft. below the dam crest. Each supported a load of 10 to 13 tons without appreciable settlement during the pouring of the rib. For lateral support, the pile groups were braced to the piers with double $\frac{7}{8}$-in. steel cables tightened with turnbuckles, using timber struts at the ends of the

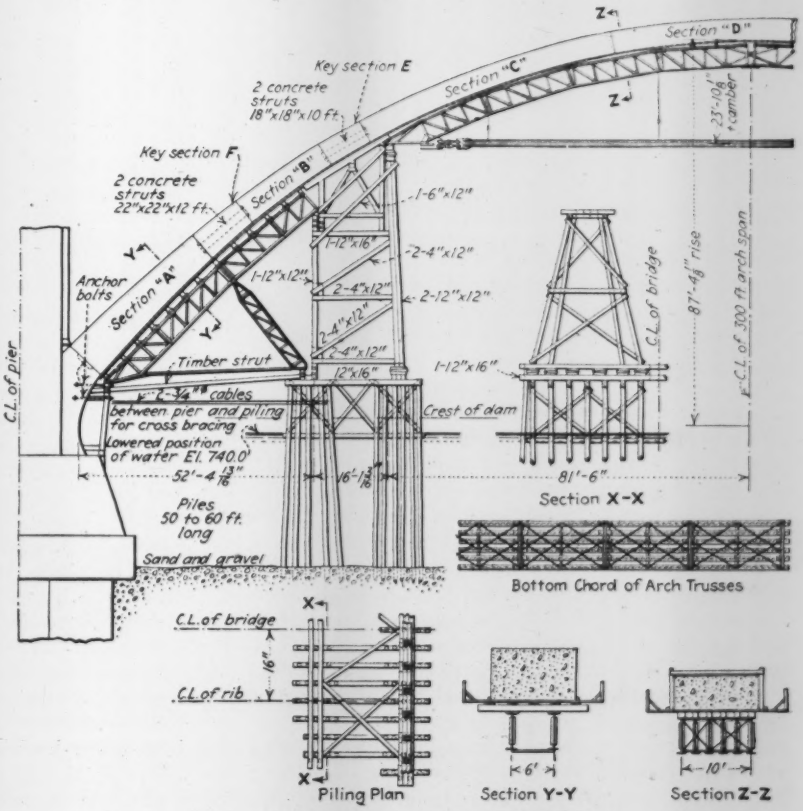


FIG. 7—DETAILS OF ARCH CENTERING

pier. At first the caps under the 161-ft. spans were of fir, later reinforced with steel plates bolted to the sides, but these were replaced by unreinforced white-oak caps, which proved satisfactory.

The centering for the middle section of the arch, with 161-ft. span, was made of six parallel trusses, built up of units approximately 20 ft. long, all bolted and with pin-connection at the crown, steel ties taking the horizontal force. On account of the unavoidable slight settlement at the timber towers, small hair cracks developed at the haunches of the first arch rib. To avoid these cracks in the other ribs, double concrete struts 22 in. square were placed in a keyway between the first and second section of pouring next to the piers. To increase the lateral stiffness, similar 18-in. concrete struts were placed in the upper keyway also. Otherwise the concreting proceeded as planned, with the last pour outside of the keyways at the crown. As expected, small cracks developed in the upper side of the concrete struts, but caused no harm to the arch ribs. After all concrete sections in the ribs had been placed, the ends of the reinforcing steel were clamped together and the four keyways poured simultaneously. In order to save time, the amount of cement was increased in the concrete for the keyways, permitting removal of the centering in fourteen days. The cement was also increased in the struts, so as to speed up the pouring of successive sections of the rib.

A sufficient amount of steel centering was furnished for only three of the six 300-ft. arch ribs. As the piers were all completed above water the easiest method to handle this centering would have been first to support one rib on the same side of the center line of the bridge in each of the 300-ft. spans and from this position move the centering sideways to the other rib in each span. This method had the great disadvantage of delaying the construction of the floor until all ribs had been poured. As speed was essential, the contractor elected to support both ribs at the same time in the east span, and one rib in the middle span. As soon as the first rib in the east span could safely carry itself, its centering was removed to the second rib in the middle span. When the first rib in the middle span could support itself, the centering was moved to the third or west span, and finally the centering of the second rib of the middle span was removed to the west span. By this method it became possible to complete the entire floor by the fall of 1926. All piers were originally designed for the method used in placing of the centering.

In lowering and moving the centering, the towers were rolled onto a barge and moved bodily, leaving the 161-ft. middle section hanging from the concrete, to be lowered later in two halves and dismantled for re-erection. The addition of the concrete struts made the centering more rigid and reduced the deflection. (See also the view in *Engineering News-Record* of Sept. 2, 1926, p. 371.)

Concrete—The concrete proportions were: For the superstructure 1 : 2 : 4, for the railings 1 : 2 : 3, and for the foundations 1:3:6 in the dry or 1:3:5 if poured under water. The caisson shells were made of 1:2:4 concrete, or 1:2:3 for parts poured under water. All coarse aggregate, except that used for the railing, was of trap rock.

Rigid control of the materials, including their grading and water content, together with the use of bulk cement, were important features of the concrete work. A fixed amount of cement was specified for the various proportions of concrete and was adhered to throughout the work. Otherwise the proportions of sand and rock were varied to conform to results obtained by field sieve tests and by tests of the moisture content of the sand. The latter was determined by frequently inundating samples and comparing the results with those obtained with dried sand. A total maximum of six gallons of water per sack of cement was found to be sufficient, even with large amounts of steel in the forms. By careful spading surfaces were obtained that are remarkably free from patches and air holes.

During every pour, lasting sometimes continuously for 70 hours, the number of batches was recorded and compared with the calculated concrete volume and amount of cement used. The average strength of all 28-day test cylinders of the 1:2:4 concrete was 2,800 lb. per sq.in. The method used for the grading and proportioning of aggregates was in accordance with recommendations of the Portland Cement Association. The entire work involved about 40,000 cu.yd. of concrete, 1,780 tons of reinforcing steel, and 90,000 cu.yd. of filling. The contract price for the bridge and approaches was $1,427,000.

Personnel—By a state law this work was in the hands of a joint bridge committee, with C. M. Babcock, state highway commissioner, as chairman. The design and construction were carried out under the direction of N. W. Elsberg, city engineer of Minneapolis, and George M. Shepard, city engineer of St. Paul, with the writer in immediate charge for the preparation of plans and the construction of the bridge. C. R. Hansen was resident engineer. The contractor was James O. Heyworth, Inc., of Chicago, with Thomas Oseth as superintendent in charge of the work and W. N. Hill as field engineer. I. F. Stern, Chicago, was consulting engineer for the contractor.

Electric Welding in Maintenance

AT THE fall meeting of the American Welding Society in Detroit, J. E. Webster, works engineer, Westinghouse Electric & Manufacturing Co., East Pittsburgh, Pa., discussed the uses he has found for electric welding plant maintenance. The following information is abstracted from his remarks: In the maintenance department there are now four operators and four portable welding sets. Maintenance work requires such a variety of tradesmen that for many years the company's steel erection has been done with carpenters; detail drawings were necessary and the steel had to go through the fabricating shop.

Since welders have been used it has not been necessary to make as accurate measurements, and hence a saving is made of the draftsmen's time and of the cost of errors formerly made in dimensioning the fabricated steel. Drawings now are necessary only to show sections to be used and welding required. A bill of material is made from the drawings. The steel is then purchased, cut to length, and delivered directly to the shop where the carpenters and welders do the complete erection. Welding operators are carpenters who have been given two or three months' training in the company's welding school. It has been found that one-third the number of men on an average erecting job should be welders in order efficiently to use the other two-thirds. By the use of welding it is estimated that one-third of the drawing room time is saved. From 10 per cent to 20 per cent in time is saved on the complete job.

Steel-Frame Repair Shop for Locomotives; C., I. & L. Ry.

Pit Tracks Served by 200-Ton Traveling Crane and Transfer Table—Heavy Column Design—Transverse Monitors

IN THE new locomotive repair shop at the main shop plant of the Chicago, Indianapolis & Louisville Ry., at Lafayette, Ind., one of the special features is the reduced height of the structure. This is made practicable by the use of the gap type of traveling crane for handling the engines. With these cranes, the bridge or crane girders are about 15 ft. apart, with a hoisting trolley on each girder, leaving the central space open so that in lifting an engine the smokestack, dome and upper part of the boiler can be drawn up between the girders, thus reducing the clear headroom required. Although this type of crane is not entirely new, it has not been used extensively and is a notable part of the equipment in the new shop. A structural feature of the building is the arrangement of the roof with high and low bays so as to form transverse monitors whose glazed sides admit additional light upon the interior of the working space. The general design is shown in Figs. 1 and 2.

The steel-frame building, now under construction, is 413x78½ ft., with no interior columns. The columns and roof trusses are spaced generally 22 ft. c. to c., but the end bays are 25 and 28 ft. wide in order to give ample working room between the engines and the walls. One interior bay of 27 ft. 8 in. conforms to the spacing of the adjacent machine shop, which originally ended at that point and had a wide end bay to provide more space between the last engine pit and the wall. The clear headroom under the trusses is 49 ft., with the crane rails 23 ft. 2 in. and 36 ft. 10 in. above the floor for the 15-ton messenger crane and the 200-ton engine crane.

On its south side the building adjoins an old machine and boiler shop 413x95 ft., while its north side fronts on the pit of an electric transfer table which serves these locomotive shops and also the car shops on the opposite side of the pit. One of the repair shop tracks extends through the machine shop and connects with the system of yard tracks. This provides for running cars directly into the shop, so that materials can be handled by the cranes.

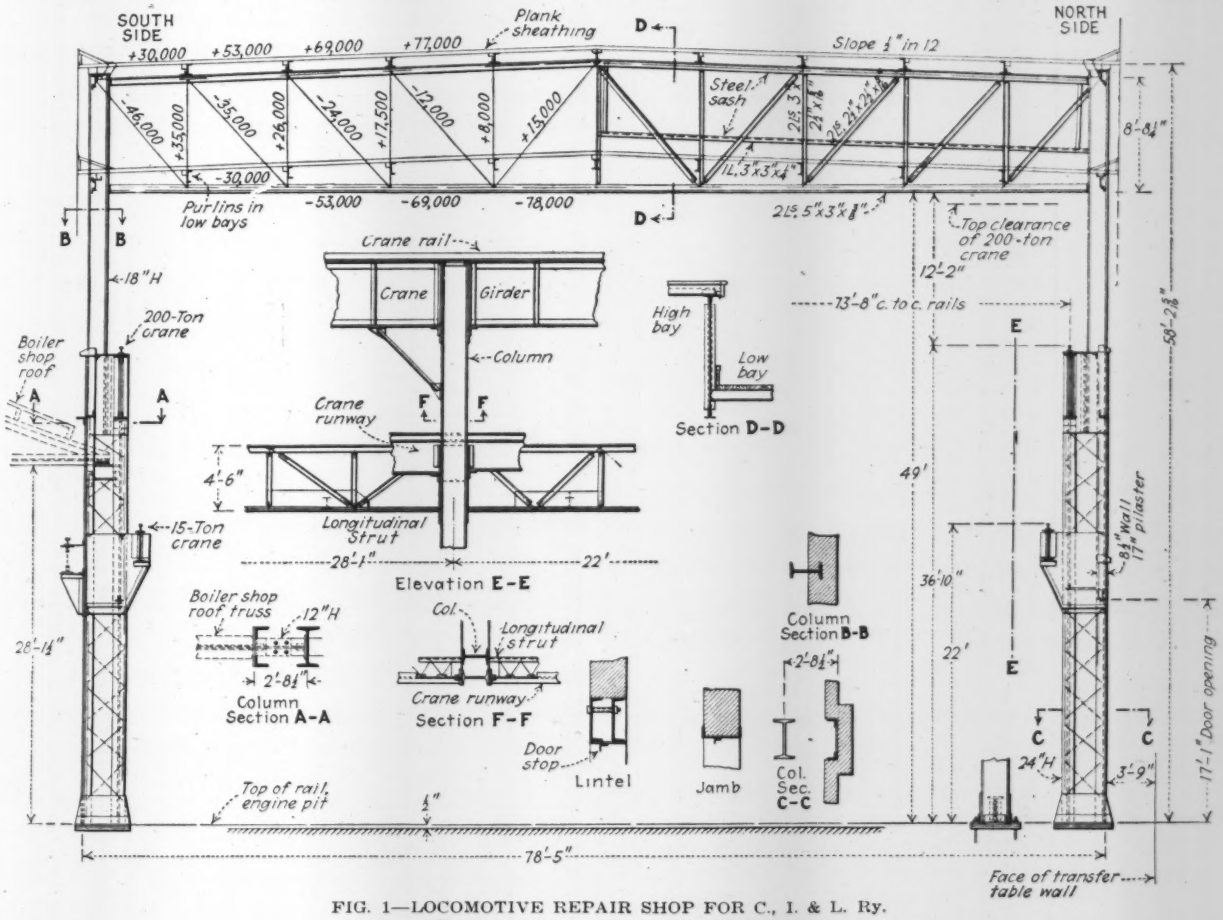


FIG. 1—LOCOMOTIVE REPAIR SHOP FOR C., I. & L. Ry.
Typical section and structural details

Concrete footings for the main columns rest on a gravel formation and are of the spread reinforced-concrete type, designed for a maximum bearing load of 6,000 lb. per sq.ft. They are 5 to 10 ft. deep, with a top pedestal 5 ft. 8 in. x 4 ft. 10 in., below which the sides flare out to the required depth. Foundation piles are not necessary. Between the footings is a 12-in. concrete wall 3 ft. deep, to retain the fill and support the side walls. Its top is level with the floor and is slotted for the rails at the doorways. Smaller footings suffice

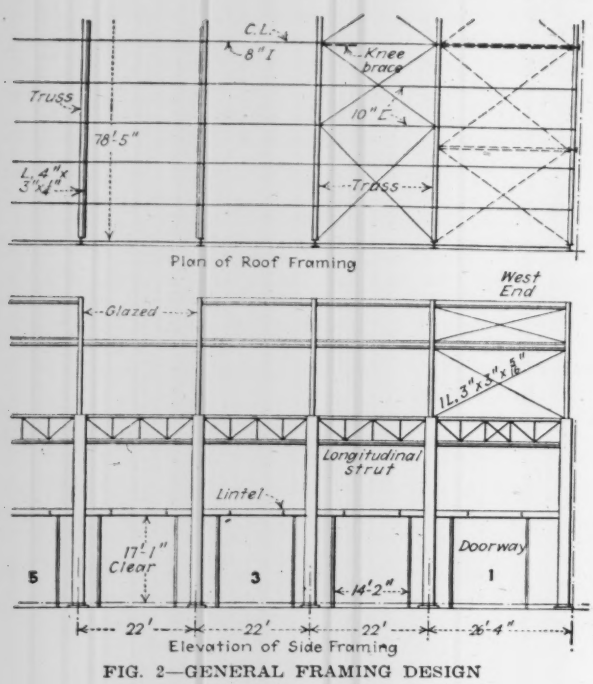


FIG. 2—GENERAL FRAMING DESIGN

for the columns in the end walls, which have to carry only the load of the wall framing. Wooden folding doors are made with four vertical hinged sections, thus avoiding the difficulty of handling large doors on windy days.

Structural Design—The two rows of side columns carry the roof trusses and the two crane runways. Some of the columns on the south side also support the roof of the old boiler shop and carry one runway for the traveling crane in that shop (see Fig. 1). Up to the height of the upper crane runway, the columns are of built-up section with a width of about 36 in. over all, transverse to the center line of the shop. The outer or wall member is a channel composed of a pair of angles and an outside cover plate, while the inner member is a 24-in. H-beam having its web parallel with the wall. Single lacing is applied on both sides of the column, except that double lacing is used on the outside of the corner columns. Thus the column section is about 18x36 in.

For the lower or 15-ton crane, the runways are 24 and 30-in. I-beams, carried on pairs of bracket plates extending the full width of the columns. These beams are held laterally by lacing bars attached to the top of a longitudinal strut between the columns. For the upper or 200-ton crane, the runways are 5-ft. plate-girders seated on top of the I-beam members of the columns. The girders have knee braces to the flanges of the I-beams, and are held laterally by plates across the tops of the columns. Above this upper runway the column consists of an 18-in. H-beam extending to the top of the roof truss and having its web at right angles to the wall, so that its outer flange is flush with the face of the main part of the column.

Where the columns carry the low roof trusses of the boiler shop, a 12-in. I-beam is fitted in the column to form the seat, its web being riveted between pairs of vertical connection angles on the cover plate and web of I-beam of the column. Behind the lower runway beams are light trusses framed between the interior I-beams of the columns. There is diagonal bracing in the end bays and in some of the intermediate bays. These same bays also have diagonal bracing between the top chords of the roof trusses. Additional longitudinal bracing is provided by the lintels, and by a line of struts carrying an upper stretch of brick wall.

The side walls are of red brick, 8½ in. thick, to a little above the lintels and again for a narrow strip in line with the upper crane runways, the latter being for architectural effect and breaking up the large surface of glass wall. The remainder of the wall is of steel sash and wire glass, fitted with pivoted panels for ventilation. The end walls are of similar construction. In the side walls, each bay has a doorway, 17 ft. 1 in. high and 14 ft. 2 in. wide. The lintels are pairs of 10-in. channels, with all flanges turned in toward the shop (see Fig. 1). The channels are connected by bolts and spacing sleeves, and have a bottom plate $\frac{5}{16}$x8¼ in. On the underside of the plate is an angle forming the door stop. Door jambs of 9-in. channels extend to the lintels. All interior steel is finished with aluminum paint.

Simple Pratt roof trusses with a top chord slope of 1 on 12 carry purlins of 10-in. channels, except that the

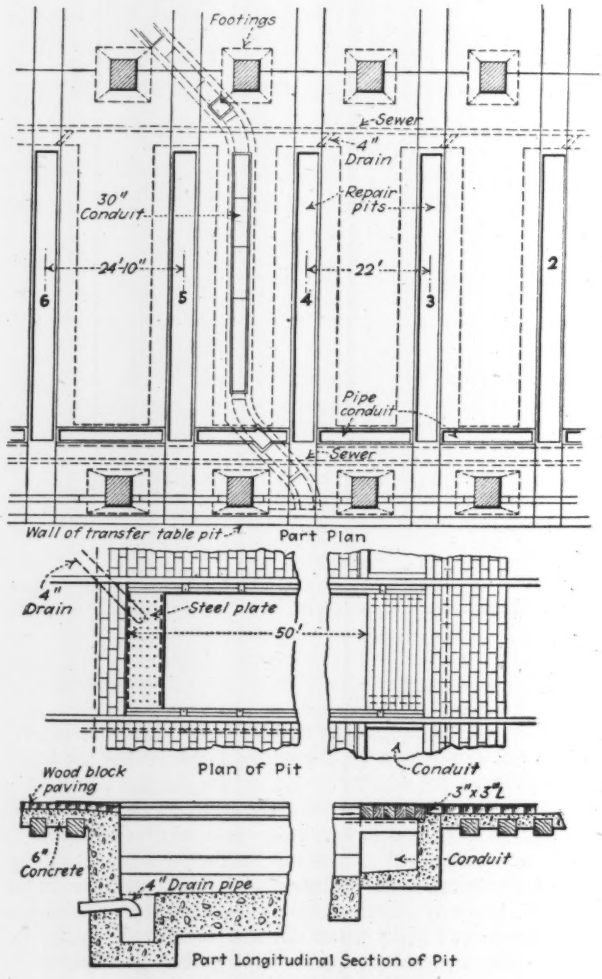


FIG. 3—ENGINE REPAIR PITS

purlin at mid-span is an 8-in. I-beam. In the high bays, these purlins are laid on top of the trusses. In every third or fourth bay, however, the roof is dropped about 8 ft., and the purlins for the low bays are riveted to the truss posts, as shown in Fig. 1. The sides of these transverse monitor roofs are mainly of steel sash and wire glass. Nailing strips bolted to the purlins carry a sheathing of tongued-and-grooved plank, upon which is applied a prepared roofing.

Repair Tracks, Pits and Floor—Engine repair pits, Figs. 3 and 4, are provided in fifteen of the stalls or bays, the tracks in the other three stalls being used as part of the adjacent boiler shop. Two of these tracks extend into the boiler shop, and another extends through the machine shop to connect with the yard tracks.

The pits, formed in reinforced concrete, are 50 ft. long, with a clear width of 4 ft. 1¼ in. and a depth increasing from 2 ft. 8 in. at the north end (transfer table) to 3 ft. 2 in. at the south end. The bottom is flat, since the engine boilers are empty, and there is much less water than in the repair pits in engine house tracks, which latter pits usually have a crowned bottom. In each pit and covered by a perforated steel plate, is a sump, with a 4-in. pipe leading into one of the sewers.

Heating and Lighting—Six unit heaters with thermostatic control will maintain a uniform temperature throughout the building, four having each a capacity of 1,027,000 B.t.u. per hour and the others each 1,122,000 B.t.u. per hour. They will be supplied from the main steam line running through the shops, reducing valves giving a pressure of 25 lb. at the heaters.

The main electric lighting system will consist of 500-watt lamps with steel reflectors. Three lamps will be suspended from the roof truss in each bay and one will be fastened to each column by a bracket located under the runway of the 15-ton crane. In addition, each track will be equipped for ten portable extension cords for the use of the various craftsmen working on the locomotives. Six of these portable lights will be connected to cast-iron receptacles in the side walls of the engine pits. A large distribution cabinet mounted near the center of the south wall will be supplemented by smaller fuse cabinets at convenient points. Practically all electric conduits will be overhead, supported on the steel framing; only

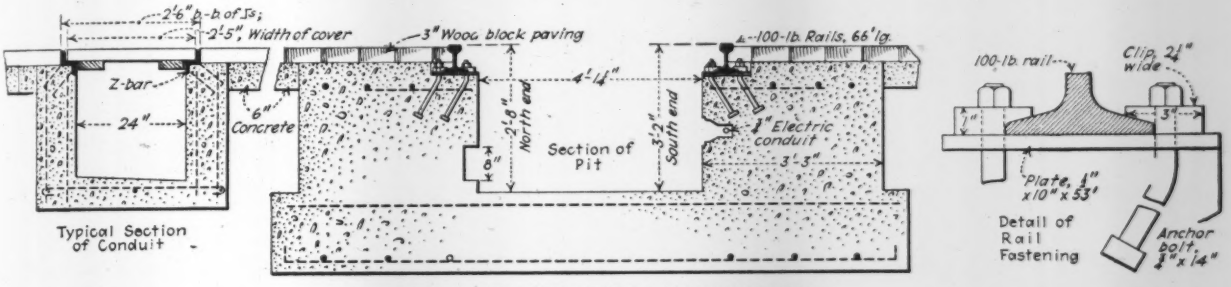


FIG. 4—PIT AND FLOOR CONSTRUCTION

one side of the pit is a continuous recess for compressed-air pipes. In the other side are pockets with electric connections for attaching the cords of portable lamps or tools.

The pit tracks have 100-lb. rails in 66-ft. lengths, laid on bedplates 10x½ in., 53 ft. long, and are secured at intervals of 2 ft. to anchor bolts by clips and nuts. The bolts are bent so as to extend back into the wall. Beyond the pits the rails are spiked to sawed ties laid in the well-tamped earth base and partly embedded in the bottom of the 6-in. concrete floor slab. Over the entire floor area this slab is paved with 3-in. creosoted wood blocks, laid on a bituminous seal coat and grouted with a bituminous composition.

At its upper end, the pit connects with a longitudinal conduit 24x24 in. for hot water and compressed air piping. This crosses a transverse conduit 30x42 in. for steam and air pipes and also connects with a separate transverse conduit 24x12 in. for hot water piping. All these conduits are rectangular, built of reinforced concrete, Fig. 4. They have 3-in. wood covers resting on Z-bars which form the top edges of the walls and are well anchored into the concrete. These covers, made of two thicknesses of plank, are in lengths of about 3 ft. and fitted with ring-bolts for ease in handling. Where the conduit forms an extension of the pit, it is covered with heavy timbers laid between the rails. Two sewers of 10 and 12-in. diameter extend the length of the building to main sewer connections. At the lower end of such conduit as is necessary to reach the engine pits will be placed in the concrete floor slab. Cables lead into the building from a transformer house, 200 ft. distant.

This repair shop was designed and built under the direction of A. S. Kent, chief engineer of the Chicago, Indianapolis & Louisville Ry. It was built by A. E. Kemmer, Lafayette, Ind., as general contractor, and was to have been completed in October, at a total cost, including equipment, of $300,000.

Study of Building Ventilation

An agreement made between the steam and gas engineering department of the University of Wisconsin and the American Society of Heating and Ventilating Engineers provides for co-operative research to determine the infiltration of air through various types of wall construction and also the amount of air which leaks into a building around the windows and doors, both with and without weather-stripping. Information of this kind is much needed by engineers who design heating and ventilating plants for large buildings. The society and the college of engineering will each furnish $1,000 annually for this work. A program of tests has been prepared by a committee headed by Prof. A. C. Willard, head of the mechanical engineering department of the University of Illinois, and the tests will be conducted at the Camp Randall shops of the University of Wisconsin under the direction of Professor G. L. Larson.

Development of Water Treatment at Indianapolis

Open Slow Sand Filters Built in 1904 and Covered in 1905-6—Rapid Filters for Enlargement 20 Years Later

By Harry E. Jordan
Filtration Engineer, Indianapolis Water Co.

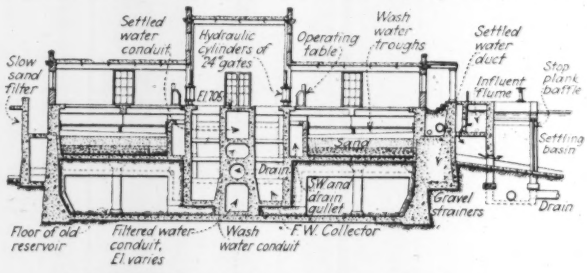
THE public water supply of Indianapolis, since the Indianapolis Water Co. first began operation in 1872, was derived first from infiltration wells and galleries, then later augmented by deep wells in the Niagara limestone. At present major reliance is upon filtered water derived from White River. All infiltration wells and galleries have been abandoned and no emergency untreated river connection has existed for many years. The deep wells still furnish part of the supply. The board of directors of the company, in April 1902, definitely approved suggestions looking toward the construction of a slow sand filtration plant. Chief among the changes since then have been the subdivision and roofing of the slow sand filter units and, some twenty years later, the addition of rapid or mechanical filters.

The period of 1900-02 was one when purification projects were under way in a number of the larger cities of the country, notably Philadelphia, Pittsburgh and Washington. At Washington filtration was investigated by the Corps of Engineers of the United States Army under the direction of Colonel Miller, and while his recommendation was for the construction of a rapid sand plant, such opposition developed in the community (led by the Medical Association of the District of Columbia) that the committee of the United States Senate appointed to investigate the question overruled Colonel Miller's recommendations. Washington, like Albany, Philadelphia, Pittsburgh and Indianapolis, built a slow sand filtration plant. Subsequently all these plants were modified upon the basis of operating experience. In each case it was found that, while the plant was able to handle very adequately raw water of normal conditions, any overload, especially of suspended matter, was reflected (in a corresponding variation) in the quality of the finished product together with a very decided lessening of the output of the plant due to surface or subsurface clogging of the sand layer.

At Indianapolis three filters were built, each having an area of 1.6 acres. These were uncovered. The relatively large size and lack of cover was brought about by the desire to take advantage of some mechanical method of removing the soiled surface sand. The first unit was completed and put in service on Sept. 23, 1904. The operation during the first winter indicated the necessity of covering. It had also been made reasonably clear that the system of mechanical cleaning which had been contemplated would not be satisfactory and that units of the size then constructed were too large. Accordingly, beginning May 30, 1905, the three filters were taken out of service in order, a central dividing wall placed in each unit, making six beds of 0.776 acre, each covered with a flat slab roof supported on cast-iron columns. This reconstruction was completed Aug. 7, 1906. The abnormal difficulties due to formation of ice in winter were eliminated and the division into smaller units produced a more even amount of filtered water.

During the year following the reconstruction of the plant an average of 11.7 m.g.d. of water was produced, the filters operating at an average of 2.6 m.g.d. per acre. During 1908, the average total output was reduced to 8.6 m.g.d. It was understood by this time that the filtration of White River water in slow sand units without preliminary treatment was producing a considerable deposition of suspended matter in the entire sand layer and that when the amount of suspended matter became quite high not only was a certain proportion of this carried through the filter but there was a masking of the biological action within the sand layer which reduced the efficiency of the unit. Investigation of the operating conditions in other cities of the country having plants of the same type indicated that this difficulty was not confined to the local situation. Consideration of the methods which could be used to overcome the difficulty resulted in the construction of a preliminary settling basin with a retention capacity approximating 48 hours' supply, to the water entering which a coagulant would be applied when required. The slow sand filtration plant at Poughkeepsie, N. Y., had adopted this method of pretreatment some years before.

At Indianapolis, after the settling basins were built, the reconditioning of the sand layer and use of coagulant when the load of suspended matter in the raw water above 30 p.p.m. resulted in a gradual increase of production. By 1913, improved methods of operation permitted the daily average output to reach 20 m.g.d. Hypochlorite of lime was first used in 1909, liquid chlorine in



CROSS-SECTION OF INDIANAPOLIS FILTERS

1916, and part-time prechlorination in addition to post chlorination in 1921. Adequate preparation of the water for filtration now permits the slow sand units to operate at an average running rate of 6.2 m.g.d. per acre, the highest rate for any slow sand plant—Blaisdell type of cleaning excepted.

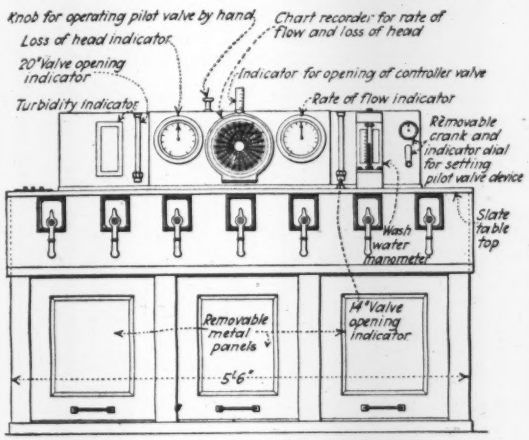
It was recognized that the original filter area of 4.656 acres could not be expected to continue to meet the growing demands of the city—even though operating efficiency was further improved. In 1924, under the direction of Carleton E. Davis, then manager of the company, an engineering group, consisting of B. J. T. Jeup, chief engineer, W. C. Mabee, designing engineer, and the writer, filtration engineer, was directed to prepare for an extension to the purification plant. Construction begun in April, 1925, was carried to completion under the direction of Howard S. Morse, the present manager of the company, and water first delivered from the new plant March 1, 1926.

In considering the type of plant to be built, there was no feeling that the slow sand plant was failing to give effective and adequate service under present day operating conditions, but it was agreed that a rapid sand plant would be more likely to meet future requirements in water-supply practice with less additional auxiliary con-

struction. Accordingly, the additional structures took the form of a 12-m.g.d. rapid sand plant—six 2-m.g.d. units—with a twin coagulation basin of 6 hr. total retention capacity.

Since filter design follows fairly well developed lines, it is worth while to describe only such features as deviate from the conventional. The first consideration was to lessen all head losses possible. The conditions under which the slow filters were located made it possible to depend upon gravity flow through the new plant. The raw water passes the plant site at El. 707 and filtered water discharges against elevations that may reach 693 or 695 as the storage reservoirs are filled.

The raw water conduit for the 12-m.g.d. plant is a 42-in. concrete pipe, the settled water line in the interbasin bank, 54 in., and the filtered water collector is 5 ft. 11 in. x 4 ft. 6 in. Instead of having the conventional pipe gallery, the structure follows somewhat the lines of some of the newer plants, with a series of superimposed ducts for filtered water, waste water, wash water and settled water occupying the middle line of the building. Connections to the filters are in the form of branch ducts of concrete, except a short cast-


Knob for operating pilot valve by hand
Chart recorder for rate of flow and loss of head
Loss of head indicator
20"Valve opening indicator
Indicator for opening of controller valve
Turbidity indicator
Rate of flow indicator
Removable crank and indicator dial for setting pilot valve device
Slate table top
Wash water manometer
Removable metal panels
5'6"
14"Valve opening indicator

FILTER OPERATING TABLE

iron special for the wash water manometer and another assembly in cast iron for the controllers.

The coagulation basin is a twin structure without cover. Each unit measures 310 x 85 ft. at the water surface. The average depth is 12 ft. The outer banks and dividing bank were formed of earth placed with tractor-drawn dump scrapers. The lining is a 4-in. reinforced-concrete slab. The water enters through a series of distributor ports at the end nearest the filters and travels in a direct line to a series of 15-in. openings at the opposite end, 21 in. below the water line. A branch collector carries the settled water to the 54-in. main return line which lies with its center line 2¼ ft. below the water level in the paved embankment between the two sections of the basin. The basin was located immediately adjacent to the filter house, primarily so the operators could easily observe the quality of the coagulated water as it entered the basin. There are no baffles. The flow is straight from the inlet to the outlet. The arrangement has given satisfaction. Ice that formed during the past winter rode up and down the 1 on 2 slope with the changes in basin level and did not freeze solid against the slopes at any time.

Careful study was given to the filter underdrain system assembly. Each filter, with a total area of 721 sq.ft., is divided into two collecting areas. Below the floor level there is an 18x31-in. branch duct for each area. These ducts lead to a main header across the front of the filter which connects in one direction to the controller and in the other to the wash-water line. On the top of the branch collector duct is bolted a ⅝-in. galvanized-steel plate running across the filter. In this, on 12-in. centers, is a series of openings into which the underdrain tees are bolted. The assembly (of two 3-in. cast-iron underdrain pipes with their tee) was made up outside the filter and then lowered and bolted into place. The assembly can be removed quickly and easily by reversing the installation process. The 3-in. underdrains on 12-in. centers are perforated with $\frac{7}{16}$-in. holes at 6-in. intervals, giving 210.42 sq.in. opening area in a 721-sq.ft. filter (20¾ x 34¾ ft.).

Above a layer of 3-in. stones laid to the top of the underdrains are seven layers of gravel with a total depth of 22 in. and 24 in. of Wisconsin sand. The distance from the sand to the wash-water trough is 24 inches.

The wash-water troughs are made of $\frac{3}{16}$-in. steel with a reinforcing angle at the overflow point. The troughs are suspended at six points by bolts and may be adjusted whenever the wash-water overflow is not even. There are seven troughs in each filter with an interspace of 43½ inches.

Much study was given to the operating table design. It was the feeling that conventional lines could well be modified. On a steel frame is placed a series of baked-on-enameled-steel panels, that are easily removed, do not warp or break, are light and easily handled. The tables are placed out over the wash-water collector that runs across the front of the filter, so that the operators can get a good view of the unit when it is being washed. Slate, instead of marble, was used for the table tops. On the top of the table are placed indicating and recording loss-of-head and rate-of-flow gages, a filtered water turbidimeter, indicators for two valves and a wash-water manometer. The inlet and drain valve cylinders stand at the operating floor level and need no indicators. Wash water is pumped to the ⅛-m.g. steel tank by centrifugal pumps that automatically refill it after washes. Measurement of the total wash water in a 24-hr. period or the replacement for an individual wash can be made by reference to the record of the 12-in. meter in the pump discharge line. It was felt that the use of meters on the main discharge line from the wash-water tank did not furnish conveniently to the operator the information as to wash-water application that the individual indicator on each table gives. The rate of wash is shown to the operator as he stands at the table by reference to the manometer connected to the wash-water line leading to the filter unit.

All connections from the table to hydraulic valves are made in ½-in. copper service pipe. The tables and their equipment were installed by employees of the department who had been selected to operate the plant after it was completed. This procedure served to familiarize them with every detail of the control equipment.

Instead of developing a coagulant plant for this unit, the structure used for the slow sand plant coagulant handling was modified. As now operating, alum is brought to the plant in company-owned trucks that unload the 200-lb. bags from the carload shipment. The material is elevated to the upper floor of the building by a continuous bag elevator, and thence trucked and dumped loose into the storage bins. There are four of

these bins, each having a capacity of 100,000 lb., but customarily filled with an even carload of 80,000 lb. As the alum is required, a fine spray of water trickles down through the lump alum and out at the bottom of the tank to the pump suction pit. The strength of this alum syrup ranges from 20 to 40 per cent and is maintained throughout the operation of the unit until less than 500 lb. of the lump alum remains on the grid at the bottom. The strong solution is lifted to a high-level storage tank by means of a water-engine-driven pump. The parts of the pump coming in contact with the strong alum and all piping connections are made of bronze. Water from the tank flows, mixing with the strong alum-solution, through a hydrometer-controlled tank to a final adjusting tank. Here, under control of conductivity apparatus, the strength of the alum solution is brought to the density at which it is fed to the raw water. Flow is controlled by a manually-operated sleeve-valve type of orifice, equipped with a recording device showing the degree of opening. Normally, the alum solution is adjusted to 5 per cent strength, but variations can be made to equalize the dilution water demand. Computation of the amount of material used is made by reference to the recorded density of the final solution and recorded opening of orifice. From a stores-issue standpoint, the records of material used are obtained as easily and accurately as by any other method except the primitive method of counting empty bags. The equipment has met demands varying from 1,500 to 30,000 lb. of alum per day, in either case requiring part time only of one man per 8-hr. shift. Plant labor places the alum in the storage-dissolving tanks and it is not handled again. Operation of the pumping and control apparatus is so simple that the single shift operator could attend to an indefinite extension of the daily output; because the adjustment of solution is an operation not essentially dissimilar when handling 1,000 or 100,000 lb. daily. Method of handling bulk material to the tanks would naturally change if the latter amount were approached.

The first year's operation of the rapid filters justified the design. Especially did the capacity and type of the settling basin demonstrate its efficiency in the character of the settled water and the length of filter runs. The maximum turbidity of the filter effluent was 0.3 p.p.m. and the average 0.17. Bacterial removal cannot be expressed in the usual terms, because of the use of chlorination as an integral part of the purification process. Not only is effluent chlorination a continuous process, but preliminary chlorination enters into the operation under certain raw-water conditions. All water from both the slow and rapid sand plants is combined and chlorinated as it enters a contact basin of 1-hr. capacity. Excess chlorine determinations are made each hour by the plant operators, who adjust chlorine application to maintain 0.1 p.p.m. residual chlorine as the water leaves the plant. The effect of this method of control is shown by the average 37-deg. figure of 7 per c.c. in the effluent water and a colon group density of 0.6 per cent of the 10-c.c. samples positive with no frequency higher than 3 of 10 portions positive.

All phases of the plant design were handled by the company's engineering staff. Settling basin excavation and fill, filter house superstructure and wash-water tank erection were done by contract. All other construction and installation of equipment were done by force account.

New Research Results on Iron and Steel From British Papers

THREE papers presented before the British Iron & Steel Institute at its annual meeting in October gave new research results and experimental data bearing on the strength of wrought iron and mild steel.

Cold Rolling and Subsequent Annealing—Prof. C. A. Edwards and Prof. K. Kuwada studied the relation of annealing temperatures and degree of cold rolling to hardness. Specimens of mild steel were given various degrees of work hardening by cold rolling in a small set of hand rollers and were subsequently annealed at various temperatures. Hardness was determined by Brinell tests. It was found that the greater the cold working, the lower is the temperature at which the removal of its effect begins; annealing tends to increase the hardness of the cold-worked material, up to about 500 deg. C. annealing temperature; the most effective range of temperature for removing the effects of cold work is approximately 550 to 650 deg. C.; and finally, in annealing between 650 and 750 deg. C., those specimens which had been subjected to a reduction of thickness of 12 to 19 per cent became distinctly softer than the virgin metal. The authors believe atomic re-orientation can be invoked to explain these results.

Repeated Tensile Impact—A new machine to produce repeated tensile impact by the action of a falling weight was described by Prof. J. H. Smith and F. V. Warnock. Tests of wrought iron were made with this machine, using various numbers of blows up to about 17,000. They conclude that there were indications of a limiting range of tensile impact energy, somewhat corresponding to the endurance or fatigue limit in repeated-stress tests. For the iron tested they found that the total energy required for fracture was constant, no matter how many repetitions were required; and the energy required for fracture was roughly the same whether the load was applied by the impact of a falling weight or by a static load. This disagrees with prior observations, and is explained by the authors as being due to their unusual care in avoiding "leaks" of energy in their machine. When they plotted the results of their repeated impact tests in a manner similar to that used for the stress-cycle diagram or ordinary fatigue tests, they obtained fairly straight lines with a definite break like that in regular endurance tests. Unlike the latter, however, this diagram appeared to show the lower segment of the line sloping instead of horizontal, but the tests were not carried to a sufficient number of cycles to be conclusive on this point. Surface finish appeared to be of little importance in their (relatively short-time) tests.

Prolonged Stress at 300 Deg.—Specimens of mild steel were tested by W. Rosenhain and D. Hanson, of the National Physical Laboratory, under steady load at high temperature (300 deg. C.) maintained for periods of over five years. The load applied was not the same in different cases, but ranged from one-third to two-thirds the ordinary tensile strength. None of the specimens failed, and none showed a marked reduction of area. However, there was a distinct increase in hardness, as shown by modified Brinell tests of the strained and the unstrained parts of the specimens. Several different microstructures of the steel were producible by suitable

heat treatment, but there was no evidence of intercrystalline cracking in the long-time tests. The specimens were not subjected to any corroding agent other than the air of the furnace in which they were kept, and the results of these tests, therefore, appear to emphasize the importance of chemical action in producing inter-crystalline fractures, as in "caustic embrittlement."

An Unusual Type of Combination Storm and Sanitary Sewer

Gooseneck Storm Sewer in Kansas City Carries Sanitary Flow in Vitrified Clay Line Bracketed onto Main Barrel

By J. F. Brown
Office Engineer for Black & Veatch and E. M. Stayton, Kansas City, Mo.

AN UNUSUAL detail of the Gooseneck Creek sewer now under construction in Kansas City, Mo., is the method used to carry the sanitary flow in a vitrified clay pipe line resting on reinforced-concrete brackets which are tied into the arch barrel of the main sewer. This detail was not described in the previous article on the design and construction of the Gooseneck sewer, *Engineering News-Record*, April 22, 1926, p. 660, because, not being poured monolithic with the main arch barrel, it had not yet been constructed.

By using this type of construction the necessity of permitting sanitary sewage to flow through the main barrel of the sewer was eliminated. The Gooseneck sewer lies, for the greater part of its length, in the bed of Gooseneck Creek. At its upper end, this sewer connects to two existing storm sewers, both of which carry sanitary sewage, discharged into them through relief connections from sanitary sewer interceptors. These relief connections are in action, however, only when the intercepting sewer is overloaded or plugged up. In addition

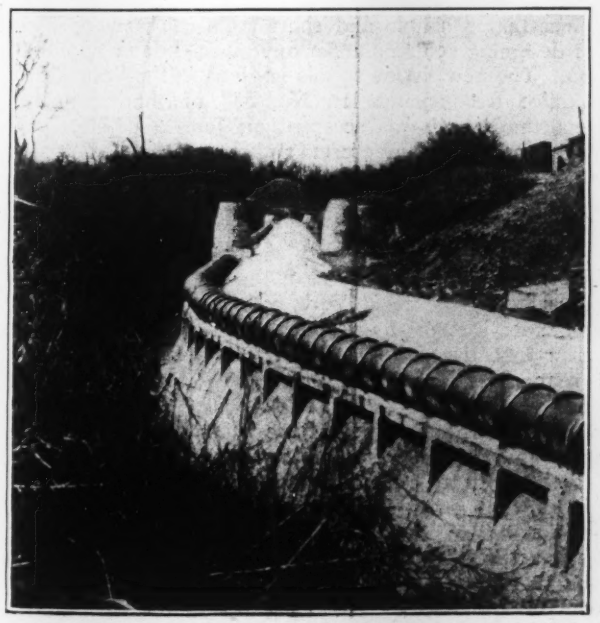


FIG. 1—SANITARY SEWER SUPPORTED ON SHELF BRACKETED TO STORM SEWER ARCH
Manholes indicate ultimate height of fill.

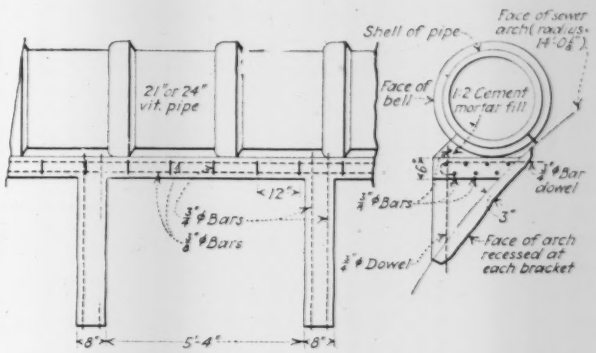


FIG. 2—DETAIL OF SHELF BRACKET SUPPORT FOR SANITARY SEWER ON STORM SEWER ARCH

to this, the entire flow of sewage from a built-up residence district of about 70 acres will be discharged into one of the present sewers at a point within 50 ft. of its connection to the new Gooseneck sewer.

The dry-weather flow of these sewers is intercepted at the inlet end of the Gooseneck sewer by means of a depressed channel below the floor, the intercepted sewage being delivered into a line of 2-in. vitrified sewer pipe laid in a trench outside of and below the wall footing of the storm sewer. The gradient of the vitrified sewer line is much flatter than that of the storm sewer, since the sanitary sewage will be discharged into an existing sanitary interceptor some 7,000 ft. below the inlet end, and at an elevation somewhat higher than the crown of the arch of the storm sewer. The gradients of the two lines, therefore, diverge until at the lower end the sanitary line crosses over the storm sewer. This gradient consideration, as well as the fact that the sanitary sewer could not be constructed in original ground except at a greatly increased cost for excavation and right-of-way, led to the conclusion to support the vitrified pipe line on a concrete shelf constructed on the arch of the storm sewer.

In constructing the storm sewer, the contractor used steel forms inside and outside for his main arch and it was therefore impossible to leave recesses or to place dowels at that time for holding the brackets. Several months after the forms were removed holes were cut by means of air hammers and dowels grouted into place to tie the brackets and shelf beam to the arch barrel. Wooden concreting forms were used. The vitrified pipe was laid, resting directly on the shelf. Joints in the pipe line are standard cement mortar joints. Manholes having concrete substructures and brick masonry superstructures are pro ided at frequent intervals to give access to the line, their substructures being built-up on the side of the storm sewer arch and bonded to it by means of steel dowels. The entire structure will remain uncovered and exposed to the weather throughout a great part of its length, since the excavation did not afford sufficient earth. However, as the low ground along the creek is developed, fill will be placed until the final grade will be 4 to 8 ft. above the top of the sewer. The total length of sanitary sewer supported on the storm sewer structures is 4,477 ft., of which 1,375 ft. is 21-in. pipe and 3,102 ft. is 24-in. pipe.

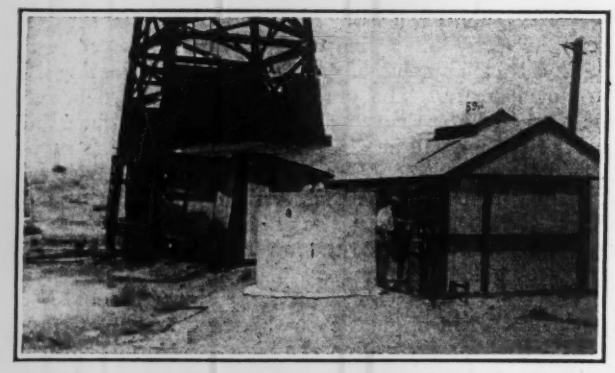
Black & Veatch and Edward M. Stayton, consulting engineers, Kansas City, made the plans and specifications for this project and are supervising its construction. The work is being done by the Torson Construction Co., of Kansas City, Kan.

From Job and Office

Hints That Cut Cost and Time for the Contractor and Engineer

Concrete Tanks for Cooling Water at Oil Field Power Plants

A FEATURE of the small gas engine power plants for the pumping of oil wells in the San Joaquin Valley oil fields in California is the reinforced-concrete tanks in which the cooling water is stored. These tanks were formerly made of galvanized iron but now the concrete tanks are preferred because of their greater durability and the fact that they can be built quickly, a large number being made by a small crew using only a few sets of forms. Although these tanks cannot be moved so easily as the lighter metal tanks they are frequently moved from one location to another. They are sometimes built at a central yard and delivered on trucks



A CONCRETE COOLING WATER TANK AT AN OIL WELL POWER PLANT

ready for service, but more frequently they are built on the site where they are to be used.

The tanks are circular in plan and the usual size of 40 bbl. has inside dimensions of 6 x 6 ft. and a shell thickness of 6 in. They are usually placed on an earth foundation but a neat cement collar is poured around the base to shut out water and where possible the foundation is located on a slight elevation to aid in drainage.

The forms consists of two bands of 3 pieces each, the sections being made of 16-gage galvanized iron bolted together with angle irons at the adjoining edges.

Protecting Exposed Water Services from Freezing

IN a paper read at the convention of the American Water Works Association last June, S. D. Bleich, assistant division engineer of the Board of Transportation, N. Y., explained a method devised to prevent freezing of water services. The following is from Mr. Bleich's paper:

The construction of ventilating chambers in the sidewalks along the subways in New York City often results in the exposure of water services. When this occurs, either a section of the ventilator is eliminated about the water service or an auxiliary 4 or 6-in. pipe is laid longitudinally between the ventilating chamber and the building line, and the service is transferred to this new line. When neither of these methods can be pursued, the water service is carried through the ventilating chamber, in which case its protection from freezing is necessary.

Extensive experimental work was done to develop a design which would prevent freezing. Tests on various insulating coverings on water pipe were made in a refrigerating plant with temperatures varying from 0 to 8.5 deg. F. When there is no flow, water in services ¾ to 1 in. in diameter will freeze in less than 3 hr. in an air temperature of 0 deg. F. Protection from an air temperature of 10 deg. F. over a period of 48 hr. was desirable, being the maximum condition of service in connections to business property, where water might not be drawn for two successive days. The base temperature of the water was taken as 40 deg. F. The tests showed that with a 1-in. pipe freezing was unavoidable unless the insulating material was 12 in. or more thick. Increasing the size of the pipe to at least 2 in. and carrying it through an insulated ferrule consisting of an inner galvanized steel pipe surrounded by 3 in. of hair felt, showed better results. The insulation then was inclosed in a steel pipe to protect the pipe and the covering from external damage. This type of insulation, consisting of three layers of 1-in. hair felt separated by asbestos paper, was adopted. The insulated ferrule is carried at least 2 ft. into the ground and is sealed at the ends with an asphalt mastic and concrete.

Spiral Reinforced-Concrete Columns

By Jacob Feld
Consulting Engineer, New York City

GENERAL acceptance of the recommendations of the U. S. Department of Commerce restricting the number of types and sizes of steel spirals manufactured has reduced the number of column possibilities to such an extent that a simple and short table can now be used for designing columns under any given code or specification. The new list of spirals is given in the "Simplified Practice Recommendation No. 53" of the Commerce Department and was accepted on Jan. 26, 1926, by a large group of manufacturers and other interested organizations and was planned to be mandatory by March 1, 1927, because of the depletion in stock of the eliminated sizes.

Under the New York City code all of the spiral sizes in the simplified list may be used—61 in all. The table opposite for the New York code gives the core sizes and areas, the spiral size, pitch, per cent and weight per linear foot, the allowable load on the spiral and concrete core together for the 1:2:4 and 1:1½:3 mixes (the only ones permitted in New York). To these load values must be added the value of the vertical rods; the minimum and maximum vertical steel for each column size, together with the area and possible rod combinations, also given. For other amounts of vertical rods, the short table at the end is used in summing the load values for any number of rods. The last column of figures is the volume of concrete for a circular column with 2 in. cover over the spiral.

Under the Joint Committee Specification, only 43 possible column combinations exist, since the ratio of vertical to spiral steel is fixed. The table shows the vertical

SPIRAL COLUMNS - NEW YORK CODE (1921)

CORE SIZE & AREA	SPIRAL SIZE	SPIRAL PITCH	SPIRAL % SPIRAL	SPIRAL WGT. PER FOOT	1:2:4 CONCRETE LOAD FOR CONC. & SPIRAL	1:2:4 CONCRETE MAXIMUM VERT. STEEL RODS	1:2:4 CONCRETE MAXIMUM VERT. STEEL LOAD	1:2:4 CONCRETE MINIMUM VERT. STEEL RODS	1:2:4 CONCRETE MINIMUM VERT. STEEL LOAD	1:1½:3 CONCRETE LOAD FOR CONC. & SPIRAL	1:1½:3 CONCRETE MAXIMUM VERT. STEEL RODS	1:1½:3 CONCRETE MAXIMUM VERT. STEEL LOAD	1:1½:3 CONCRETE MINIMUM VERT. STEEL RODS	1:1½:3 CONCRETE MINIMUM VERT. STEEL LOAD	SPIRAL SPACING	SPIRAL SPACING FOR 1 FT.
12" 113.0"	¼"	1⅝"	1.01	3.9	102	6-1"	32	4-⅝"	8	114	6-1"	30	4-⅝"	8		0.79
	⅜"	1⅜	1.96	7.6	145	8-⅝"				150	4.52"		1-1.30"			
	½"	2¼	0.89	3.4	140	8-1"	56	5-⅞"	15	160	8-1"	53	7-⅞"	14		
	½"	1⅜	0.66	4.5	153	7-1⅛"		7-⅞"		174						
16" 201.0"	⅜"	2¼	1.22	8.4	199					219						
	⅜"	1⅜	1.87	10.0	219					239						1.4
	⅜"	1⅜	1.69	11.6	236					256	8.06"		2.01"			
	½"	2¼	1.97	13.4	258					279						
	¼"	2	0.49	5.3	219	13-1"	88	7-⅜"	82	250	8-1⅛"	83	7-⅜"	21	1.5	
	½"	1½	0.66	7.0	240	10-1¼"		5-⅞"		271					#/ft each.	
	½"	2¼	0.98	10.5	282	8-1¼"				313					for Bost.	
20" 316.0"	⅜"	1⅞	1.26	13.5	319					351						22
	⅜"	1½	1.67	15.7	342					373						
	½"	2⅜	1.43	18.3	338					369						
	½"	2¼	1.74	18.7	376					408	12.56"		3.14"		1.2	
	½"	2	1.96	21.0	404					435					#/ft each;	
	¼"	1⅝	0.50	7.8	317	18-1"	127	10-⅜"	32	362	18-1"	120	10-⅜"	30		
	⅜"	2¼	0.73	11.3	358	14-1¼"		8-⅞"		408					for 4.50	
	⅜"	1⅜	0.98	15.2	402	12-1¼"		6-1"		449						
	⅜"	1½	1.23	18.9	449					494						
24" 459.0"	½"	2⅝	1.24	19.2	451	NOTES:				496						
	½"	2¼	1.45	22.4	488	VERT. STEEL IS				534	LOADS ARE IN					31
	½"	1⅜	1.74	26.9	541	FOR ALL SPIRALS				586	KIPS (1000 #)					
	½"	1⅝	2.02	31.0	588	OF SAME CORE.				633						
	⅝"	3	1.70	26.2	534					578	18-1"		2.52"		3 Sp. Spacers per Spiral —	
	⅝"	2⅝	1.94	30.1	581					622						
28" 645.0"	½"	1⅜	0.51	10.7	433	16-1¼"	173	10-⅜"	43	495	20-1⅛"	163	10-⅞"	41		
	⅜"	2¼	0.74	15.6	490	20-1⅛"		8-1"		553						
	⅜"	1⅝	0.97	20.6	547			6-1"		610						
	½"	2⅜	0.97	20.5	547					610						
	½"	2¼	1.24	26.2	614					675						
	½"	1⅝	1.89	31.6	675					737						4.3
	⅝"	1⅝	1.78	36.2	732					794						
	⅝"	3	1.46	30.6	670					730					3	
	⅜"	2¼	1.75	36.8	739					800	2.46"		6.16"			
	⅜"	2¼	1.95	40.9	793					835						

3/1/27 JACOB FELD

SPIRAL COLUMNS - NEW YORK CODE (1921)

CORE SIZE & AREA	SPIRAL SIZE	SPIRAL PITCH	SPIRAL % SPIRAL	SPIRAL WGT PER FOOT	1:2:4 CONCRETE LOAD FOR CONC. & SPIRAL	1:2:4 CONCRETE MAXIMUM VERT. STEEL RODS	1:2:4 CONCRETE MAXIMUM VERT. STEEL LOAD	1:2:4 CONCRETE MINIMUM VERT. STEEL RODS	1:2:4 CONCRETE MINIMUM VERT. STEEL LOAD	1:1½:3 CONCRETE LOAD FOR CONC. & SPIRAL	1:1½:3 CONCRETE MAXIMUM VERT. STEEL RODS	1:1½:3 CONCRETE MAXIMUM VERT. STEEL LOAD	1:1½:3 CONCRETE MINIMUM VERT. STEEL RODS	1:1½:3 CONCRETE MINIMUM VERT. STEEL LOAD	SPIRAL SPACING	SPIRAL SPACING FOR 1 FT.
	⅜"	2¾"	0.50	13.8	563		225	18-⅞"	56	643	24-⅛"		8-1"			
	⅜"	1⅜	0.73	20.2	637			11-1"		718		212		53		
	½"	2¼	0.99	26.9	721	32-1"		8-1"		802						
32"	½"	2	1.23	33.6	798	26-⅞"				878						
804.0"	½"	1⅜	1.51	41.3	888	21-1⅛"				970						67
	⅝"	2⅜	1.46	40.0	872					952						
	⅝"	2¼	1.70	46.8	950					1030						
	⅝"	2	1.92	52.6	1021		34.2"	8.04"		1102	32.2"		8.04"			
	¾"	2¼	0.74	17.0	708		303	13-1"	71	810	26-⅛"		8-1⅛"			
	¾"	1⅜	0.75	26.2	814			10-1"		915		286		67		
	½"	3	0.72	25.2	802	32-1⅛"		8-1⅛"		903						
	½"	2¼	0.98	33.6	900	26-⅛"				1000						
36" 1018"	¾"	1½	1.24	43.2	996					1113						35
	¾"	1⅛	1.65	50.4	1100					1200						
	⅝"	2¼	1.51	52.7	1125					1225						
	⅝"	2	1.70	59.0	1204					1308						
	⅝"	1⅜	1.95	67.5	1302	40.7"		10.18"		1404	40.7"		10.18"			
	⅜"	2¼	0.50	21.0	880		351	16-1"	88	1005	32-1⅛"		8-1⅛"			
	⅜"	1⅛	0.73	31.6	994			13-1"		1120		331		83		
	½"	2⅜	0.74	32.0	1000	40-⅛"		10-⅛"		1125						
40" 1257"	½"	2	0.98	42.0	1122	32-1⅛"		8-1⅛"		1248						
	¾"	1⅝	1.21	51.8	1237					1361						
	⅝"	2	1.53	65.7	1398					1523						
	⅝"	1⅜	1.75	74.8	1508					1630						
	⅝"	1⅛	2.04	87.5	1632	50.2"		12.57"		1760	50.2"		12.57"			

FOR 1:2:4 MIX @ 700#/□" LOADS FOR VERTICAL STEEL. FOR 1:1½:3 MIX @ 900#/□"

SIZE NO	½"	⅜"	¾"	1"	1"	1¼"	1¼"	SIZE NO
1	2	3	4	6	7	9	11	1
2	4	6	8	12	14	18	22	2
3	6	9	13	17	21	27	33	3
4	9	13	17	22	28	36	44	4
5	11	16	21	28	35	44	55	5
6	13	19	25	33	42	53	66	6
7	15	22	30	38	49	62	77	7
8	17	25	34	44	56	71	88	8
9	19	28	38	50	63	80	99	9
10	21	31	42	55	70	89	110	10

FOR 1:2:4 MIX - @ 700#/□"

	½"	⅜"	¾"	1"	1"	1¼"	1¼"	
	2.0	2.9	4.0	5.2	6.6	8.4	10.3	1
	4.0	5.8	8.0	10.4	13.2	16.7	20.6	2
	6.1	8.8	11.8	15.6	19.8	25.1	31.0	3
	8.1	11.7	15.9	20.7	26.4	33.4	41.2	4
	10.1	14.6	19.8	25.9	33.0	41.8	51.6	5
	12.1	17.5	23.8	31.1	39.6	50.1	61.9	6
	14.1	20.5	27.8	36.2	46.2	58.5	72.2	7
	16.2	23.4	31.8	41.4	52.8	66.9	82.5	8
	18.2	26.3	35.7	46.6	59.4	75.1	92.8	9
	20.2	29.2	39.7	51.8	66.0	83.5	103.1	10

FOR 1:1½:3 MIX - @ 900#/□"

3/1/27 JACOB FELD

REINFORCED CONCRETE COLUMNS WITH SPIRALS JOINT COMMITTEE SPECIFICATION (1924)

COL DIAM. & AREA	SPIRAL SIZE	SPIRAL PITCH	SPIRAL % SPIRAL	VERT. RODS	LOAD FOR COLUMN 2000# MIX	LOAD FOR COLUMN 2500# MIX	LOAD FOR COLUMN 3000# MIX
12" 113"	¼"	1⅝	1.01	6-1"	146	157	167
	⅜"	2¼	0.49	9-⅞"	168	183	197
16" 201"	⅜"	1⅝	0.66	7-1"	196	211	228
	⅜"	2¼	1.22	8-1¼"	302	320	343
	⅜"	1⅜	1.87	8-1¼"	356	378	402
20" 314"	¼"	2	0.49	8-1"	265	285	310
	½"	1¼	0.66	7-1⅛"	306	330	356
	⅜"	2¼	0.98	8-1¼"	397	425	459
	⅜"	1⅜	1.26	13-1⅛"	485	518	550
	⅜"	1¼	1.67	12-1¼"	557	589	628
	½"	2⅜	1.43	12-1¼"	543	577	612
24" 453"	¼"	1⅝	0.50	9-1"	383	415	450
	½"	2¼	0.73	9-1¼"	488	505	545
	⅜"	1⅝	0.98	12-1¼"	571	618	657
	⅜"	1¼	1.23	15-1¼"	684	729	778
	½"	2⅝	1.24	15-1¼"	688	736	786
	½"	2¼	1.45	17-1¼"	791	840	893
28" 616"	½"	1⅜	0.91	10-1⅛"	525	568	617
	½"	2⅛	0.74	12-1¼"	683	694	748
	⅜"	1⅜	0.97	16-1¼"	772	825	890
	½"	2⅜	0.97	16-1¼"	772	825	890
	½"	2¼	1.24	20-1¼"	937	1005	1070
	½"	1⅝	1.89	24-1¼"	1107	1165	1240
	⅝"	3	1.67	24-1¼"	1090	1155	1230

COL DIAM. & AREA	SPIRAL SIZE	SPIRAL PITCH	SPIRAL % SPIRAL	VERT. RODS	LOAD FOR COLUMN 2000# MIX	LOAD FOR COLUMN 2500# MIX	LOAD FOR COLUMN 3000# MIX
32" 804"	⅜"	2¾	0.50	13-1⅛"	680	737	796
	⅜"	1⅜	0.73	15-1¼"	832	895	967
	½"	2¼	0.99	20-1⅛"	1020	1090	1170
	½"	2	1.23	26-1¼"	1215	1295	1380
	½"	1⅜	1.51	31-1¼"	1460	1545	1645
	⅝"	2⅜	1.46	31-1¼"	1415	1500	1600
36" 1018"	⅜"	2½	0.49	13-1¼"	854	924	1000
	⅜"	1⅜	0.75	20-1⅛"	1070	1155	1240
	½"	3	0.72	20-1¼"	1045	1125	1215
	½"	2½	0.96	26-1⅛"	1265	1350	1455
	½"	1⅜	1.24	33-1¼"	1540	1645	1760
	½"	1¼	1.65	38-1¼"	1780	1890	2000
	⅝"	2¼	1.51	40-1¼"	1845	1955	2080
40" 1257"	⅜"	2½	0.50	20-1⅛"	1080	1150	1245
	⅜"	1⅛	0.73	24-1¼"	1300	1400	1508
	½"	2½	0.74	25-1¼"	1310	1415	1515
	½"	2	0.98	32-1⅛"	1585	1695	1815
	½"	1⅝	1.21	40-1¼"	1875	1990	2145
	⅝"	2	1.56	50-1¼"	2350	2490	2695

SPECIFICATIONS: % SPIRAL = ¼% to 1½%. Pitch is less than ⅙ Core Diam.
% VERT. RODS = 4 TIMES (% SPIRAL) = p.

LOAD ON COL. = A (500 + 8000 p)(1 + 14 p) for 2000# MIX.
A (550 + 10000 p)(1 + 11 p) for 2500# MIX
A (600 + 12000 p)(1 + 9 p) for 3000# MIX.

LOADS IN KIPS, ARE FOR CONC. SPIRAL & RODS.

JACOB FELD 3/1/27

REINFORCED CONCRETE SPIRAL COLUMNS - CHICAGO CODE.

DIAM.	SPIRAL SIZE	SPIRAL PITCH	SPIRAL % SPIRAL	VERT. RODS MINIM.	VERT. RODS MAXIM.	2000# MIX LOADS CONC. & SPIRAL	2000# MIX VERT. RODS MIN.	2000# MIX VERT. RODS MAX.	2500# MIX LOADS CONC. & SPIRAL	2500# MIX VERT. RODS MIN.	2500# MIX VERT. RODS MAX.	3000# MIX LOADS CONC. & SPIRAL	3000# MIX VERT. RODS MIN.	3000# MIX VERT. RODS MAX.
	¼"	2"	0.49	9-¼"		186	12		226	12		265	12	
20" 314"	¼"	1½	0.66	9-¼"	16-¼□"	196	19	175	236	19	172	275	19	168
	⅜"	1½	1.26	9-⅜"		231	27		271	27		310	27	
	⅜"	1⅛	1.87	10-⅜"		243	30		283	30		322	30	
	¼"	1⅝	0.50	10-⅜"		268	21		325	21		382	21	
24" 452"	⅜"	1⅛	0.98	10-⅜"	23-¼□"	309	31	252	368	30	247	423	30	242
	⅜"	1½	1.23	10-⅜"		330	42		387	41		444	41	
603"	½"	2¼	1.65	11-⅜"		349	46		408	46		463	45	
	¼"	1⅜	0.51	11-⅜"		367	24		444	23		521	23	
28" 615"	½"	2⅛	0.74	11-⅜"		394	34		471	33		548	33	
	⅜"	1⅝	0.97	11-⅜"	32-¼□"	420	46	350	497	46	344	574	45	337
665"	½"	2¼	1.24	13-⅜"		451	55		528	54		605	53	
	½"	1⅜	1.69	12-1"		481	66		558	65		635	64	
	⅜"	2⅜	0.50	13-⅜"		477	28		578	28		679	27	
32"	½"	1⅞	0.73	13-⅜"		512	40		613	40		714	39	
	½"	2¼	0.99	13-⅜"	41-¼□"	581	55	448	652	54	446	753	53	432
	½"	2	1.23	13-1"		587	72		688	70		789	69	
804"	½"	1⅝	1.51	13-1"		629	91		730	89		831	88	
	⅜"	2⅜	1.46	15-1"		622	82		723	81		824	79	
	⅜"	2½	0.89	14-⅜"		603	43		731	42		757	42	
36" 1018"	⅜"	1⅞	0.75	18-⅜"		652	49		780	49		806	48	
	½"	3	0.72	18-⅜"	52-¼□"	646	49	569	774	49	559	800	48	548
	½"	2½	0.98	17-⅜"		692	72		822	70		846	69	
1018"	½"	1⅜	1.24	16-1"		746	88		870	86		900	85	
	½"	1¼	1.65	15-1"		788	105		916	103		940	101	
	⅜"	2¼	1.51	18-1"		797	108		927	103		981	101	
	⅜"	2½	0.50	15-⅜"		748	46		903	46		981	45	
40" 1257"	⅜"	1½	0.73	15-⅜"		800	63		957	62		1015	61	
	½"	2½	0.74	15-⅜"	64-¼□"	803	63	700	960	62	687	1018	61	674
1257"	½"	2	0.98	15-1"	17-¼□"	859	82		1018	81		1074	79	
	½"	1⅝	1.21	15-1"		913	105		1070	103		1128	101	
	⅝"	2	1.53	15-1¼"		988	133		1145	130		1203	128	

f'c = ¼ ultimate. For 2000# Mix, n = 15
fs = n fc. " 2500 " 12
" 3000 " 10

Value of spiral = 2½ times equivalent vert steel.

Spiral ½ to 1½%
Vert Rods ½ to 8% but not less than spiral percentage

JACOB FELD 3/1/27

rods which may be used with each spiral. The loads are for the total combination of concrete, spiral and rods, for the three mixes recognized by this code. The restriction of 1½ per cent maximum spiral volume eliminates the use of some standard spirals.

Under the Chicago code, only 32 of the standard spirals may be used because of the restrictions of ratio between pitch and column diameter. In this is none of the spirals less than 20 in. in diameter. The table gives the list of spirals and the loads for the combination of each of the three permitted mixes of concrete with the spirals. The minimum and maximum rod combinations and the load values and cross-section areas of each are also given for each column. The minimum per cent of vertical steel depends upon the per cent of spiral; the maximum per cent is fixed.

It must be noticed that the specified material in the "Simplified Practice Recommendation" is hot-rolled rod and that the manufacturers of cold-drawn wire will probably continue to supply all the eliminated as well as the standard sizes. However, in the wire material, the spiral sizes are wire areas and not full section areas.

The carrying capacity of any spiral concrete column can easily be determined from these tables. For any given load, the proper combination of concrete, spiral and rods is also found with very little work.

Construction Kinks Used in Building Natural Gas Plant

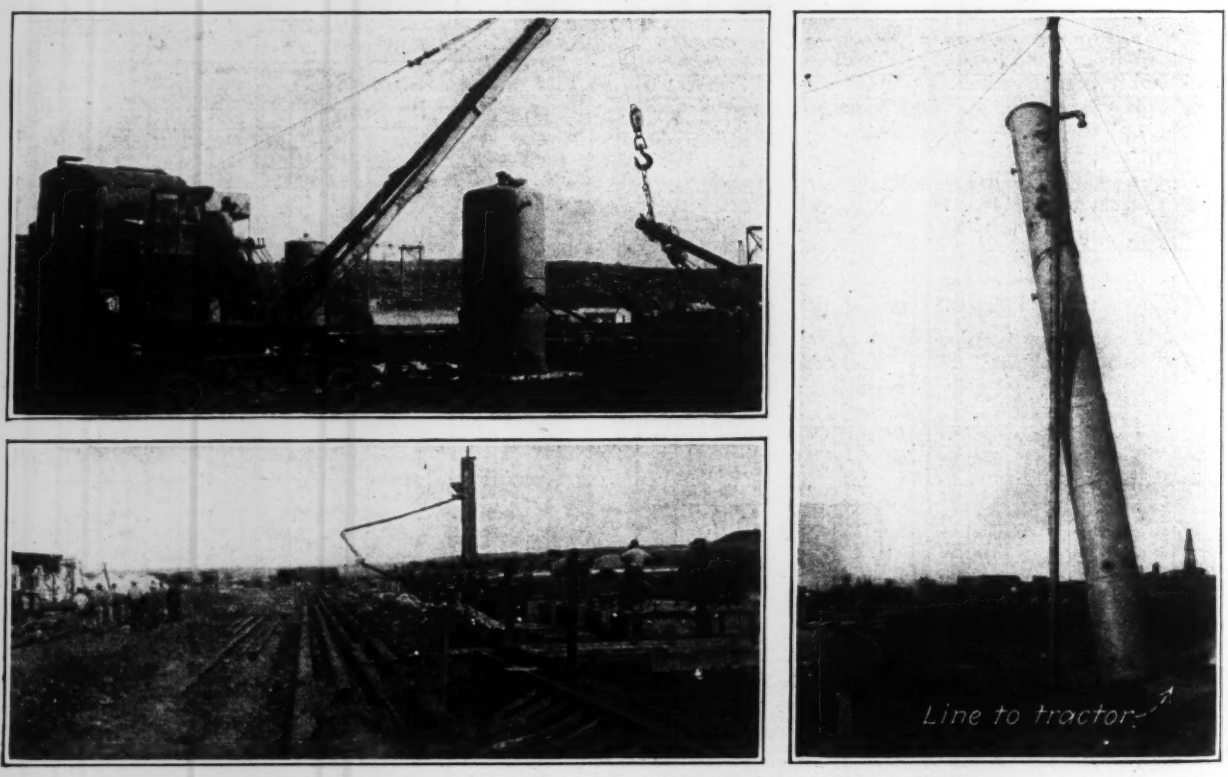
CAREFUL planning and the use of modern construction equipment have placed a 50,000,000-cu.ft. casing-head gasoline plant into operation in the Texas Panhandle 10 to 15 days sooner than would otherwise have been possible. This saving is based on the time required to construct a similar plant under the same conditions but using none of the kinks developed on the second work. Both plants were built by the Phillips Petroleum Co. in the oil field around Borger, Texas. A conservative estimate of loss of gas, rich in gasoline content, on account of plant idleness is $5,000 per day, so that speed in completion is a first requisite.

From Job and Office

Hints That Cut Cost and Time

Construction work is mainly a problem of concrete placement in the foundations and engine and compressor bases and the erecting of structural steel buildings, tanks and piping. Most of the results have centered around the use of a 5-ton crawler crane and a 10-ton crawler tractor. The crane was substituted for slow hand methods of blocking shafts and flywheels for the 165-hp. engines into position. The old method often required a half day, while the crane was able to finish the work in 10 min. When the crane was not available a steel frame moving on rollers lifted the heavy pieces of machinery. Even this, however, was slow and costly.

Small pieces of equipment, such as pumps and motors, were handled directly from motor trucks into the final location in the building. Structural steel buildings of a portable type are used for all plant housing needs. By attaching every second purlin to each roof truss and raising the assembly into position on the columns by means of the crane boom a rapid erection schedule is maintained. In the construction of the plants it is often necessary to install various sizes and shapes of tanks in elevated position where they must be connected with pipes and valves. On account of the rapid nature of



UP-TO-DATE CONSTRUCTION METHODS USED IN BUILDING GAS PLANT

Upper left: Crawler crane raising ginpole. Below: Tower and chutes concreting foundations and bases for battery of compressors. Right: Heavy crawler tractor raising 2,300 lb. tank. The tractor is not visible in the photograph.

From Job and Office

For Contractor and Engineer

gasoline plant construction it is often the case that these tanks cannot be delivered from the manufacturer at the time they are desired. As piping cannot be delayed it has now become the practice to install pipe and other equipment immediately. When the tanks arrive these are hoisted into place by the crane, thus eliminating unnecessary delay. Such tanks vary in weight from 1,000 to 9,000 lb.

One use made of the tractor is shown in an accompanying view. The tank is 50 ft. high and 4 ft. in diameter and is filled with steel shavings, weighing 2,300 lb. Formerly a team and later a winch on a light tractor were used, but both of these methods proved slow and extremely dangerous since from the time the tank leaves the ground until it finally rests on its foundation the weight is on the pulling line. The heavy tractor (not shown in the illustration) raised it to position by a steady pull. The tractor also replaced crowbar operation in moving engine cylinders onto the blocks.

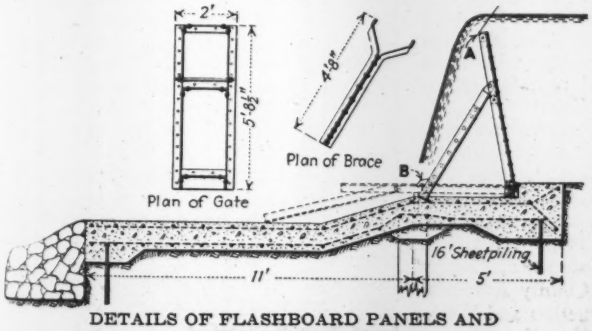
Complete assembly of heavy pipe layouts on the ground and raising to position with a hand winch on a ginpole saved much time over the old method of assembly piece by piece while hanging in the air. Steel concreting forms were developed for exhaust stacks and tank saddles, and modern chuting replaced wheelbarrow haulage wherever possible.

A. W. Riney is superintendent of construction while Paul Endacott is in charge of all field engineering work. A. H. Riney is chief engineer of the company.

Diversion Dam Consists Chiefly of Collapsible Units

By A. Kanekeberg
Public Service Company, of Colorado

IN PROVIDING a circulating water supply from the Platte River for the Lacombe steam plant at Denver, owned by the Public Service Co. of Colorado, a diversion dam was used which consists chiefly of a concrete apron with sheet piling cutoff walls surmounted by flashboards designed for operation while being overtopped. A flashboard design of some sort was required because of the wide variation in stream flow on this stream. Within



DETAILS OF FLASHBOARD PANELS AND THEIR FOUNDATION

the last ten years the variation has included a low flow of 15 sec.-ft. and a maximum flow of 8,800 sec.-ft.

The bed of the stream contains 8 ft. of loose sand and gravel overlying a thin layer of shale. It was decided to make the foundation in the form of a thick, reinforced-concrete apron with steel sheet piling cutoff walls at up- and down-stream edges and to use wood piling at 6-ft. intervals as a support for the flashboards, as shown in the accompanying drawing. The flashboards and the braces were made from heavy angle iron and 1¼-in. steel plates, being spaced with ½-in. clearance between adjoining panels. The leakage is not important except in very dry seasons and then prepared wood battens are placed over the joints to make them watertight.

For operating the flashboards a steel bridge was erected above them consisting of two 58-ft. spans carrying

DIVERSION DAM ON THE PLATTE RIVER WITH FLASHBOARDS IN SERVICE

a railroad track for the manually operated hoist. The hoist includes a steel boom with cable and drum arranged to swing on a turntable mounted on a small flat car.

The flashboards are operated by two men in the following manner: With the hoist in place the steel boom is extended upstream and the end of the cable that passes through a pulley at the outer end of the boom is connected to point *A* on the flashboard panel that is to be lowered. With the cable exerting a pull at the angle indicated, the weight on this panel is relieved and the panel brace can then be released manually by lifting at the point *B*. The panel is lowered gradually by slacking away on the hoist. Operation in this manner has been found flexible enough to comply with the requirement that the stream flow shall never exceed a depth of 6 in. over the full width of the flashboard panels in erect position.

Job and Office Notes

Well Points Were Used for Draining Water-Bearing Sand in driving the 10-ft. tunnel for the Clifton Ave. sewer system at Chicago. These well-points were driven horizontally into the face of the heading. They were connected to a small electric pump by means of hose, and the water was delivered to the surface of the ground by a similar pump. A low air pressure, not exceeding 2 lb. per sq.in. was used in this tunnel work. Owing to the character of the material, the cover being all sand, it was impossible to maintain sufficient pressure to keep the water back without the use of well points, as no shield was used in driving the heading. This combined use of light air pressure and well-points proved very satisfactory. It was devised by Mr. Dillich, superintendent of the American Sewer & Drain Construction Co., which had the subcontract for the tunnel under the White Construction Co., general contractors. The sewer is being built under direction of C. D. Hill, chief engineer of the Board of Local Improvements, Chicago.

News of the Week

Current Events in the Civil Engineering and Contracting Fields

Danville, Va., Bridge Arch Ring Collapses While Being Poured

Formwork Believed Supported on Ledge Rock in Industrial Tail-Race Channel Fails

By George C. Stone
Consulting Engineer, Altavista, Va.

THE west arch ring of the south (shore) span of the new reinforced-concrete bridge at Danville, Virginia, collapsed about 2:30 p.m. on Oct. 31 while the concrete was being poured. Eight workmen were more or less seriously injured, and others scratched or bruised.

The bridge is a new reinforced-concrete arch structure of seven spans, being built on the site of the old steel bridge which was destroyed by fire on June 30 (see *Engineering News-Record*, July 21, 1927, p. 103). The structure is of open spandrel type, with twin arch rings. Pouring of the concrete rings was started in the middle of the stream and advanced toward both shores, the westerly line of rings being poured first.

Forms for the south or shore span were supported on a braced falsework of round timber posts, supposed to be seated on the ledge rock of the river bed. At the point of failure the posts were standing in the water channel of an industrial tail-race, about two feet deep and rather swift. The report given out after the accident was to the effect that the posts in question had settled vertically, thus tearing loose the bracing and throwing down the load.

At the time of collapse, that particular ring was being poured and the concrete was nearly all in place. The first sign of failure was a slow but perceptible slumping in or flattening of the shore end of the ring, about the quarter point of the span. Warning was given, and most of the workmen succeeded in getting off in time, the others riding the ring as it went down. The form barrel was torn apart a little to the north of the crown, and the north end of the green concrete ring was left standing. This north end was promptly finished off with a bulkhead construction joint, to be built to as soon as the collapsed formwork could be re-erected.

The writer was unable to examine the wreck until after part of it had been cleared away, but is of the opinion from what appeared then that in erecting the formwork what was in reality loose shingle and accumulated debris in the tail-race had been mistaken for bed rock. The posts carrying the formwork of the arch ring were seated on this loose material in the swift water of the tail-race, and when the increasing load of green concrete had reached a certain point apparently the loose material under one or more of the posts crushed, squeezed out, or tilted and caused the feet of the posts to slide, thus bringing undue stress upon other posts and so resulting in a progressive failure up to collapse.

Record Rainfalls Cause Heavy Damage in New England States

Vermont Worst Sufferer—Transportation and Communication Lines Crippled —Water Supplies and Sewerage Facilities Affected

RAINFALL and floods exceeding all records for seventy-five years or more devastated a large part of New England last week, causing perhaps 150 deaths, heavy property losses and cutting off large areas from communication by railway, highway or wire. Rapidly rising streams and consequent torrential flow caused much destruction, but submersion affected many towns seriously. The heavy rainfall area extended from eastern Maine across New Hampshire and Vermont to Lake Champlain, with lighter rain but heavy floods reaching southwards through a large part of Massachusetts, northern Rhode Island and eastern Connecticut. There were overlaps at the North over the Canadian border into eastern Quebec and western New Brunswick, and on the West into eastern New York from Whitehall, on Lake Champlain, down the Hudson to Albany and vicinity. Precipitation of from 4 to 6 in. was common, with maximum thus far reported of 9½ in. in 24 hours at Somerset, Maine.

The storm thus was of phenomenal character, in view of the great area of high rate of precipitation. The downpour came on rain-soaked ground, which resulted in rapid and large runoff.

The Connecticut and eastern Lake Champlain drainage basins were the chief sufferers, but in Maine the Kennebec was in flood, as also the Androscoggin from Berlin, N. H., to Auburn and Lewiston, Me.; in New Hampshire and Massachusetts, the Merrimac, Manchester to Lowell and Lawrence in Massachusetts; in Rhode Island, the Blackstone, from Worcester to Woonsocket; in Connecticut, the Naugatuck and other streams at the east.

Vermont was the worst sufferer. Its worst hit river valley was the Winooski, from Barre through Montpelier, the state capital, Waterbury, Bolton, Richmond, Winooski-Burlington to Lake Champlain, north of Burlington. In this valley occurred the 86 deaths out of the total of 114 for Vermont and 135 for all New England, reported up to Nov. 7. Of two more northerly tributaries of Lake Champlain, the Lamoille wrought damage at Morrisville, Cambridge and on to Milton; and the Missisquoi at Richford, on the Canadian border, Berkshire and more westerly towns. Still another Vermont city next the Canadian line, Newport, on Lake Memphramagog, suffered from floods in the Black River. Towards the southern part of Vermont, on a Champlain tributary, Rutland was a heavy sufferer, while farther south damage was done at Ludlow and other places, as also at Bellows Falls, the Connecticut River junction point for the Ludlow stream.

Practically the entire length of the Connecticut River was at a record-breaking flood stage, the chief towns from north to south being Wells River Vt., and Woodsville, N. H.; Norwich,

County Highway Officials' Program Announced

The annual meeting of the County Highway Officials Division of the American Road Builders Association will be held Jan. 9-13, 1928, at the Hollenden Hotel, Cleveland. The meetings of the standards committees will begin on Jan. 9 and the final reports will be made Jan. 12, which is County Highway Officials' Day at the 25th annual convention of the American Road Builders' Association. The standards committees include those on Surveys and Planning, County Highway Construction, Rural County Highway Maintenance, Urban County Highway Maintenance, County Highway Legislation, County Administration, County Construction and Maintenance Equipment, and County Finance.

The officers of the County Highway Officials Association are Thomas J. Wasser, president; Charles M. Upham, secretary, and James H. MacDonald, treasurer.

Vt., and Hanover, N. H.; White River Junction, Windsor, Bellows Falls and Brattleboro, the latter at the Vermont-Massachusetts line. In Massachusetts heavy damage was done at Northampton, Holyoke, Springfield and West Springfield; in Connecticut, Hartford was the chief sufferer. At Holyoke, Mass. (drainage area 8,400 sq.mi.), a height of 14.75 ft. was reached on the dam about 5 p.m. Saturday, compared with a maximum of 12.7 ft. in the 1869 flood. At Hartford, the peak of the flood at midnight, Nov. 6-7, was 29 ft. above mean low water, or 10 in. below the level of the flood of 1854.

The extent to which dams gave way to the floods is uncertain yet but throughout the larger part of the flooded area those built according to sound engineering design seem to have stood. It is certain that in the main, the streams rose because of the heavy rains on water-soaked areas reaching brooks and rivers already above normal due to preceding rains. Water supplies were cut off, sewers flooded, and gas and electric service stopped or badly crippled.

Due to scores of wrecked bridges, the washout of hundreds of culverts, and damage to roadbed, railroads and highways in Vermont and New Hampshire were largely out of commission for several days and cannot be got into full service before winter or spring. Telephone and telegraph service was crippled for a time. Radio gave partial but increasing service. Airplane aid was not available for two or three days and on coming into gradual use was greatly hampered by the condition of the relatively few landing places available, which were soft.

The general flood conditions in Vermont are reported by Arthur D. Butterfield and Louis B. Puffer, professors in the engineering college of the University of Vermont, as follows:

At the present time it is impossible to give anything except the most meagre information. Highway and railroad bridges have been carried away and each river valley is isolated with no definite information as to what has actually happened in a neighboring valley. Telephone and telegraph communication has been interrupted and information has been brought in some cases by couriers who had to make their way on foot.

The following information concerns itself with only the flood effects in the vicinity of Burlington and Winooski on the Winooski River. The flood was occasioned by a very heavy rain that commenced Wednesday night and continued almost without interruption for nearly two days. During this time practically 6 in. of rain fell or over one-sixth of the annual rainfall. At other parts of the state 7 in. of rain fell.

P. & A. Photos.

FLOOD DRIFT AT BECKET, MASS.

The Winooski River is about 60 miles in length and has a drainage area of about 1,000 sq.mi. Between Burlington and Winooski Village, the river discharges through a channel about 270 ft. wide with a normal depth of around 6 ft. In this channel the water rose to the height of the flooring of the steel highway, some 15 ft. above normal stage and carried the bridge away, at 2:50 p.m., Friday. The maximum flood stage occurred about five or six o'clock in the afternoon, at which time the height of the flood was about 0.5 ft. higher than at the time the bridge was carried away.

Approximate computations, which can be checked later when the water has gone down and measurements made possible, indicate a flow of approximately

P. & A. Photos.

RAILWAY BRIDGE AT MONTPELIER WRECKED BY FLOOD

100,000 sec.-ft. The maximum stage lasted for several hours and the foundations of one building and the roadway on the Burlington side were dynamited in order to widen the channel and help relieve the pressure on the American Woolen Co.'s mills on the Winooski side of the river and which were flooded into the second story. By Saturday morning the river level had dropped about 3 ft. and from then on fell rapidly.

The actual loss in bridges and dams cannot be told at the present time. Burlington has been practically isolated from Waterbury, Montpelier and other places up the river valley. Railroad connections are impossible for all railroad bridges on both railroads out of Burlington have been swept away, also telephone and telegraph communications destroyed. What little information that has come as to losses, indicate a heavy property and life loss. Rumors would tend to show that some dams have been washed out. As far as known, however, not many, if any, large dams have gone out, but many smaller dams have been lost. The losses affecting the engineering side have been chiefly in the bridges, both railroad and highway, broken water supply connections which may result in some cases in a serious situation, and in the destruction of concrete highways under process of construction.

It is known that Waterbury, Montpelier, Rutland and some smaller towns have temporarily, at least, lost their water supply.

Bridge Destruction

Bridge Damage on the Winooski—The Winooski River in the first 25 mi. above its mouth, just north of Burlington, was crossed by eleven bridges. The effect of the flood on each of these bridges, taken in order going up stream was briefly as follows:

No. 1. Rutland R.R. bridge at the mouth of the river, three spans of about 150 ft. each, through single track trusses, collapsed early Friday afternoon before the crest of the flood was reached. Probably failed from underscoring of the piers in the river.

No. 2. Highway bridge, old covered wooden Howe truss, two 150-ft. spans built on grade. Apparently unharmed.

No. 3. Central Vermont R.R. bridge between Burlington and Winooski. Two spans of about 150 ft. each, decked single track trusses recently rebuilt and strengthened. Failed through underscouring of pier which settled vertically 4 or 5 ft., allowing one span to slip off and several hours later overturned, wrecking the other span.

No. 4. Burlington-Winooski steel highway bridge. Two 135-ft. through truss spans carrying street railway tracks, highway, gas mains, power and telephone cables. Failed shortly before the crest of flood, from flood pressure of water and drift wood caught on the lower chords. This bridge was located about 450 ft. below one dam and 150 ft. above another and the current had a velocity of at least 20 ft. per sec.

No. 5. Lower Central Vermont R.R. bridge at Winooski Gorge. Two spans single track deck trusses high above the river just below a 30-ft. dam. Pier carried away by force of water and bridge destroyed.

No. 6. Upper Central Vermont R.R. bridge at Winooski Gorge. Single span decked truss, just above the dam. Unharmed although the water reached the lower chord about 25 ft. above normal level.

No. 7. Highway bridge on Lime Kiln Road. Single span concrete arch, high above the river. Unharmed.

No. 8. Highway bridge at Essex Junction. Single span, steel through truss about 300 ft. below a 50-ft. dam and 50 ft. above the river. Bridge safe but washed around abutments. Water apparently came within about 5 ft. of lower chord.

No. 9. Central Vermont steel bridge above Essex Junction. Three spans, recently rebuilt, single track. Weakened but still standing.

No. 10. Central Vermont R.R. bridge. No report.

No. 11. Highway bridge below Richmond. Very old wooden combination arch and truss, single span covered bridge. Still standing but highway washed out around both abutments so that at present it cannot be used.

The following reports were received by H. S. Knowlton, Boston representative of *Electrical World:*

D. W. Jardine, general manager of the Burlington Vermont Light and Power Co., stated that the rainfall at Burlington totaled 5.62 in. in thirty hours and that the estimated maximum discharge of the Winooski River was 100,000 sec.-ft. The dams and plants of this company are undamaged except for water soaking. Steam plants are now carrying the loads and the hydroelectric stations will probably be in complete operation in about a week.

Rivers in Eastern Maine ran high, but no flood damage was reported by Frank Silliman of the Bangor Hydro Electro Corp. The maximum flow of the Androscoggin River was 58,000 sec.-ft. and of the Kennebec 50,000 sec.-ft. The maximum rainfall for the storm period on the Androscoggin was 5.25 in. and on the Kennebec 3.43 in. The maximum rainfall on the Merrimac River at Garvins Falls was 4.2 in. The water rose over the dam 9.6 ft. and the maximum flow was 67,000 sec.-ft.

No permanent damage was done to the plants or lines of the Turner's Falls Power and Electric Co., according to F. L. Hunt, chief engineer.

The Connecticut River flow at Turner's Falls was 170,000 sec.ft. The rainfall at Turner's Falls was 4.2 in. All machines at Cabot ran through the flood period. Water rose outside the station to a level 2 ft. above the operating floor, but by the use of barricades of sand bags, and by pumping, no harm was done.

A. S. Jordan, Connecticut Light and Power Co., Waterbury, Conn., stated that practically normal service was maintained on the entire system. Damages were nominal and the maximum rainfall was 5.23 in. The crest of the flood passed Springfield at 5 p.m. Sunday, and Hartford Monday at 2 a.m. At Hartford a flow of approximately 200,000 sec-ft. was registered.

Destruction to Dams

From all that can so far be learned no important public dams went out. Chittenden held above Rutland, Vt.; the Bristol dam on the Pemigewasset was holding at last accounts; Somerset and Davis Bridge reservoirs were only two-thirds full after the flood on the Deerfield watershed. Where first-class engineering design and construction obtained, it appears that the dams have held, and so far as can be learned, those which went out were chiefly small ones with private owners, or old small public utility dams which probably should have been rebuilt.

Dams which failed include: Dam above Glen station, Rutland, Vt.; Becket, Mass., Wheeler dam and Lyman reservoir dam; St. Johnsbury, Vt., dam; dam at Sharon, Vt.

A gravel crib dam above the Glen station at Rutland failed, flooding this plant of 1,500-kw. rating. The Mendon pipe line serving this system burst and was open for 300 ft. Seven inches of rain fell at Rutland Wednesday and Thursday. The Chittenden reservoir dam in the mountains about seven miles above the city held about $6,000 worth of crushed rock, having been placed on the downstream slope in the past summer for additional strength.

On the Twin State system a dam was lost at Bennington, Vt.; the canal bank was partly washed away at Hoosick Falls, N. Y., and the West Dummerston station at Brattleboro was under water, with the dam apparently intact.

The generating plant of the Bradford (Vt.) Electric Light Co., Inc., was about 5 ft. under water, but service was restored to sixteen communities by Monday. The Wilder plant of the International Paper Co. was under 9 ft. of water, but the new 1,600-hp. development under construction at Canaan, N. H., and 40 miles of line tying this station into Lancaster were uninjured.

The New England Power Co.'s plant at Vernon had 3 ft. of water on the bus room floor and 16 ft. of water over the crest of the dam. The normal head on this station, which is rated at 30,000-kw., and is the first plant built by the New England Power organization, is 34 ft. The tail water at Vernon rose to a height of 5 ft. above the dam crest.

Prof. H. K. Barrows, professor of hydraulic engineering at Massachusetts Institute of Technology, sends the following illuminating data:

The storm causing this flood disturbance was first noted on the weather map of Wednesday, Nov. 2, with low area in Southern Florida. On Thursday the low center had proceeded to Cape Hatteras, on Friday morning the low center was in Central Vermont and on Saturday morning somewhat north of Quebec. The weather map of Friday shows a low trough extend-

ing from about Montreal to southern Connecticut, with lowest barometer about 29 in. and approximately parallel isobars running northwest to southeast showing a storm disturbance of wide area extending from the North Atlantic states through into Maine and Canada, with isobars 29.8 at these limits. It was therefore a fairly typical Atlantic Coast storm.

Precipitation—Such data as are available for the total rainfall in this storm are shown in the accompanying table. It appears that a maximum of 10 in. or more was reached at stations in Central and Southern Vermont and a rainfall of 4 in. or more probably extended over most of Vermont and New Hampshire, Western Massachusetts and Eastern New York.

The rainfall at Somerset, 9½ in. in 24 hours, was exceptional and appears to be the greatest recorded at any station for this period of record in New England. In the storm of October, 1869, a rainfall of 12.3 in. was measured at Canton, Conn. This was, however, for a period of 48 hours, the 24-hour record not being known. The extent of the highest rainfall amounts is not at present known and may have included a considerable area in Northern Vermont and New Hampshire as well.

Flood Run-Off

On the Connecticut River the flood exceeded all previous records. At the Vernon, Vt., plant of the New England Power Co., where the drainage area is 6,300 sq. miles, a height of 16 ft. was reached on the dam early Saturday morning, the estimated flow at this time being about 155,000 sec.-ft. or about 25 sec.-ft. per square mile.

At Turners Falls, with a drainage area of 7,100 sq. miles, a maximum height of about 13½ ft. on the dam was reached about 9:00 a.m., with an estimated flow of about 180,000 sec.-ft. or about 25 sec-ft. per square mile.

At Holyoke, Mass., drainage area 8,400 sq. miles, a height of 14.75 ft. was reached on the dam about 5:00 p.m. Saturday, with an estimated flow of about 152,000 sec.-ft. or 18 sec.-ft. per square mile.

For comparison, the maximum heights reached at the Holyoke dam for a number of noteworthy floods have been as follows:

Year	Feet
1854	10.5
1862	12.6
1869	12.7
1902	11.9
1913	12.7

The Deerfield River had a relatively small flood damage, due to the effect of the Davis Bridge and Somerset Reservoirs of the New England Power Co. At the beginning of the flood these reservoirs were both approximately two-thirds depleted of storage and the effect of the flood was to add about one-third to the amount of water stored. This corresponded to approximately 2.5 billion cu.ft. of water, or nearly 6 in. depth over the 182 square miles of drainage area above the Davis Bridge Reservoir.

The gage height at the Charlemont gaging station of the U. S. Geological Survey as a maximum was 13.38, for which the corresponding discharge is probably about 35,000 sec.-ft., as compared to a stage of 15.7 reached on July 8, 1915, with corresponding discharge of 45,000 sec.-ft., which represented the highest water in recent years on the Deerfield. Without the storage in the reservoirs of the New England Power Co. it is likely that the discharge at Charlemont would have been perhaps 20,000 sec.-ft. larger, making a total of 55,000 sec.-ft., which would have meant a gage height of about 18 ft.

The flood of 1915, which occurred before the construction of the Davis Bridge Reservoir, caused very material damage on the Deerfield River, and it is safe to say that if this reservoir had not been built the present flood would have far exceeded that of 1915 in respect to damage and perhaps disaster.

While unusual high water prevailed on the Merrimac River apparently no records were reached or exceeded on the main river. At Lawrence about 9 ft. of water maximum went over the dam, with a discharge of about 70,000 sec.-ft. In both 1852 and 1896 the height of water at the Lawrence dam was over 4 ft. greater than that of the present flood. Here, again, there was some reduction in flood due to water being held back in Lake Winnipesaukee and possibly other reservoirs.

On the Pemigewasset River at Plymouth a gage height of 28 ft. was recorded as a maximum, which would correspond to a discharge of about 35,000 sec.-ft. or some 57 sec.-ft. per square mile. Here, again, while fairly large, the flow is not exceptional.

PRECIPITATION DURING STORM OF NOV. 3-4, 1927

Station	Total Rainfall for storm inches	Remarks
Vermont		
Somerset	10.0	9.5 in. in 24 hours.
Vernon	4.0	
White River Jc.	6.0	
Davis Bridge Plant	5.0	Near Readsboro.
Sherman Plant	4.0	" "
Burlington	5.62	3.75 on Nov. 3.
Northfield		
Silver Lake Plant	6.5	Near Salisbury.
New Hampshire		
Concord	4.0	
Mass.		
Pittsfield	4.75	
Holyoke	4.94	
Lawrence	2.30	
New York		
Albany	3.84	
Canada		
Montreal	2.50	

R. R. Marsden, dean of the Thayer School, and professor of civil engineering, Dartmouth College, furnished *Engineering News-Record* with the following summary of conditions in New Hampshire:

The present flood conditions in New England exceed in severity those of the record year of 1869. Between the early morning hours of Thursday, Nov. 3, and 11 a.m. Nov. 4, the precipitation at Hanover, N. H., was 6.36 in. On Oct. 4 and 5, 1869, it was 6.07 in. In 1913, another flood year, the highest stage was reached on March 27. The precipitation over a period of four days was 3.69 in. and the high runoff was due to the accumulated snow. The mean annual precipitation in this region is about 40 in.

At White River Junction, five miles south of Hanover and at the confluence of the White and Connecticut rivers, the maximum stage was reached on Friday, Nov. 4, when the gage read 38 ft. A previous maximum of 30 ft. was recorded for the 1913 flood. A normal height for this season of the year is about 10 ft. Between 4 p.m. Nov. 3 and 8 a.m. Nov. 4 the rise in the river was 30 ft. The maximum stage in the Connecticut River at Hanover, however, was not reached until the morning of Nov. 4, nearly 24 hours after the White River peak had passed. The maximum flood probably carried approximately four or five times as much water as during normal maximum high water.

At Sharon, Vt., twelve miles from the mouth of White River, the concrete dam of the Sharon Power Co. was destroyed and released a large amount of pondage. No information is available as to the flood height at that point.

It is impossible at present to estimate the damage done. In a distance of 25 miles, however, there have been destroyed or seriously injured five railroad bridges, two important highway bridges, long stretches of highway and railway, as well as telegraph and telephone lines. The four lines of railroad out of White River Junction have all been completely paralyzed.

Some rainfall records, supplied by Robert E. Horton, consulting hydraulic engineer of Voorheesville, N. Y., follow:

Housatonic, Mass. (Monument Mills)

Date	In.
Nov. 2	0.70
Nov. 3	4.93
Nov. 4	0.08
	5.71

Can gage 12 in. square mounted on top of powerhouse at "Furnace" one-half mile above Housatonic. Records taken at 7 a.m.

Stockbridge, Mass.
(Stockbridge Water Co.)

Date	In.
Nov. 2	0.86
Nov. 3	4.95
Nov. 4	0.10
Total	5.91
Nov. 5	0.01

Gage at pumping station 3 mi. above Housatonic gage. Records taken at 8 a.m.

Dalton, Mass. (Dalton Power Co.)

Nov. 2	2.00	(taken at 10 a.m.)
Nov. 3	2.00	(taken at 8 a.m.)
Nov. 4	2.51	(taken at 8 a.m.)
	6.51	for 32 hours.

Total rain as above given probably all right but reading for Nov. 3 apparently an error, though gage reader maintained this was the record.

Papers state no rainfall record is available at North Adams during the storm. Williamstown, about 5 mi. away, registered 6.11 in. This was taken at Williams College and (the newspaper states) "this was the largest rainfall in North Berkshire for the period, 36 hours."

Springfield *Republican*, Nov. 5: "The precipitation in the 24-hour period between 8 a.m. Thursday and 8 a.m. Friday reached 6.16 in. at the Borden Brook reservoir of the Springfield water system and, including Wednesday night's rain, the precipitation of the whole storm at that point was 6.63 in.

"The records of five rain gages, four of them belonging to the water department and one to the city engineer were:

	Nov. 2	Nov. 3-4	Total
Burden Brook	.47	6.16	6.63
Mundale Filters	.80	5.61	6.41
Provin Mountain	1.32	2.87	4.19
City Eng. (City Hall)	..	..	4.27
Ludlow Reservoir	.66	3.58	4.24

N. Y. Contractor Given $20,000,000 Abyssinian Irrigation Concession

Concessions for a $20,000,000 irrigation development have been granted to the J. G. White Engineering Corporation, New York, by the Abyssinian government. The project consists of a highway connecting the capitol Addis Ababa with Lake Tsana, the source of the Blue Nile, and a dam at the mouth of the Lake. The water level of the lake will be raised only 5 to 6 ft.

The reservoir so formed will function in flood control and the water will be sold to the British Government and used to irrigate the territory on each side of the Blue Nile for a distance of 400 miles. The Abyssinian government will receive a royalty on the water sold during a period of years, after which time title will pass to Abyssinia.

Construction will take three to five years and will be done entirely by the J. G. White Engineering Corporation. American machinery and equipment will be landed at Jibuti on the Gulf of Aden near the Red Sea. The French-owned railway will transport it to Addis Ababa, where highway building will start. Materials for the dam will be trucked to the site. Sand, gravel and timber will be taken from nearest supplies, while cement will be bought in the open market.

East Bay Cities Vote Bonds for Water Distribution System

Nine cities on the east side of San Francisco Bay on Nov. 1 approved by a majority of more than 8 to 1 the $26,000,000 bond issue with which it is proposed to acquire or construct a local distribution system for the Mokelumne water supply project. Ballots were cast by about one-fourth of the total number of voters registered with results as follows:

	For	*Against*
Oakland	26,897	2,693
Berkeley	8,090	1,020
Alameda	3,881	925
Albany	673	35
Emeryville	306	11
Piedmont	1,083	129
San Leandro	747	87
Richmond	1,675	100
El Cerrito	211	19
Total	43,563	5,019

Under the terms of the measure approved, the bond issue will provide funds that can be spent either for purchasing such properties of the East Bay Water Co. as are desired or for building an independent distribution system, whichever the board of five directors of the East Bay Municipal Utility District decides is in the best interests of the community.

Supreme Court Decision on Grade Crossing Accidents

As noted in *Engineering News-Record*, Nov. 3, p. 718, the U. S. Supreme Court laid down a "standard of conduct" for persons driving upon railroad tracks at grade crossings. The question is considered of sufficient interest to warrant publication of the entire decision, herewith:

The deceased, in this case, in crossing a railroad track relied upon not hearing the train which caused his death or any signal, and took no further precaution. The Court held that he did so at his own risk.

On writ of certiorari to the Circuit Court of Appeals for the Sixth Circuit.

Mr. Justice Holmes delivered the opinion of the court. The full text follows:

This is a suit brought by the widow and administratrix of Nathan Goodman against the petitioner for causing the death by running him down at a grade crossing. The defence is that Goodman's own negligence caused the death.

At the trial the defendant asked the court to direct a verdict for it, but the request and others looking to the same direction were refused, and the plaintiff got a verdict and a judgment which was affirmed by the Circuit Court of Appeals, 10 F. (2d) 58.

Goodman was driving an automobile truck in an easterly direction and was killed by a train running southwesterly across the road at a rate of not less than 60 miles an hour. The line was straight, but it is said by the respondent that Goodman "had no practical view" beyond a section house 243 feet north of the crossing until he was about 20 feet from the first rail, or, as the respondent argues, 12 feet from danger, and that then the engine was still obscured by the section house.

He had been driving at the rate of 10 or 12 miles an hour but had cut down his rate to 5 or 6 miles at about 40 feet from the crossing. It is thought that there was an emergency in which, so far as appears, Goodman did all that he could.

Train Has Right-of-Way

We do not go into further details as to Goodman's precise situation, beyond mentioning that it was daylight and that he was familiar with the crossing, for it appears to us plain that nothing is suggested by the evidence to relieve Goodman from responsibility for his own death. When a man goes upon a railroad track he knows that he goes to a place where he will be killed if a train comes upon him before he is clear of the track. He knows that he must stop for the train not the train stop for him.

In such circumstances it seems to us that if a driver cannot be sure otherwise whether a train is dangerously near he must stop and get out of his vehicle, although obviously he will not often be required to do more than to stop and look. It seems to us that if he relies upon not hearing the train or any signal and takes no further precaution he does so at his own risk.

If at the last moment Goodman found himself in an emergency it was his own fault that he did not reduce his speed earlier or come to a stop.

It is true as said in Flannelly v. Delaware & Hudson Co., 225 U. S. 597, 603, that the question of due care very generally is left to the jury. But we are dealing with a standard of conduct, and when the standard is clear it should be laid down once for all by the courts. See Southern Pacific Co. v. Berkshire, 254 U. S. 415, 417, 419.

Judgment reversed.

Road Commission's Right To Construct and Operate Toll Bridges Questioned

The right of the Arkansas Highway Commission to construct and operate toll bridges has been questioned in a suit filed Oct. 26 by W. B. Conners of Augusta, Ark., in the Chancery Court at Little Rock. Conners contends that the act authorizing the construction and operation of toll bridges is void for three reasons: That it conflicts with the section of the constitution which gives jurisdiction of roads and bridges to the county judges; that it violates the section of the constitution which prohibits the state from loaning its credit; and that the bill was improperly passed by the legislature. He would enjoin the State Highway Department from employing engineers to make plans for a proposed toll bridge across the White River near Augusta and from constructing the bridge and issuing or selling bonds, and from operating the bridge.

The proposed bridge would be an important link on the Little Rock-Memphis highway via Bald Knob and Augusta. Several similar bridges are planned for other points in the state. The highway commission plans to divert the tolls collected to pay the cost of the bridge and later operate the bridges free of tolls.

Fire Test Made in Holland Tunnels at New York

A fire test to determine the efficacy of the fire fighting apparatus and of the ventilating system in the Holland Tunnels under the Hudson River at New York was made on November 3. An old touring car was taken into the tunnel, six gallons of gasoline put in the gas tank and another gallon was poured over the upholstery. Then, just as the fire was set, the gas tank was punctured so that gas from it was spilling about the roadway to simulate a condition which would exist in a collision in which a gas tank was ruptured.

As soon as the fire was set, a traffic officer rang the alarm which called the emergency truck into the tunnel and set the traffic signals at stop, and immediately joined two other traffic officers in taking hand extinguishers from the recesses in the tunnel walls to fight the fire. The fire was under control in about two minutes time and was completely out in three and one-half minutes. Four of the 5-gal. Foamite extinguishers were used in putting out the fire. The emergency fire apparatus was not called into service in this test.

but subsequently gave a demonstration of its effectiveness in a second fire in which a quantity of waste was put into the wreck of the old car and again wet down with gasoline.

Smoke from the fire rose rapidly and spread along the ceiling of the tunnel and disappeared through the exhaust ports. At no time was there any objectionable amount of smoke down in the tunnel roadway, even up on the sidewalk the air was sufficiently clear to permit seeing ahead past the fire through the tunnel. By the time the fire was out, the smoke had practically all disappeared from the tunnel. The ventilating system was working at normal capacity during the test.

Personal Notes

J. V. NEUBERT, engineer of maintenance-of-way of the New York Central Lines, Buffalo and East, was appointed, effective Oct. 1, chief engineer, maintenance-of-way, of the New York Central R. R. Mr. Neubert is a graduate of Pennsylvania State College. He began service in 1899 with the New York Central & Hudson River R. R. Succeeding years brought promotions through the positions of assistant division engineer, division engineer at Albany, engineer of track, Buffalo and East, to engineer of maintenance-of-way, Buffalo and East.

WILLIAM A. MURRAY, formerly engineer of track, New York Central Lines, Buffalo and East, is appointed engineer of maintenance-of-way for this section of the New York Central Lines, succeeding J. V. Neubert. Mr. Murray entered the service of the railway in 1900 and served in the positions of bridge inspector, assistant supervisor of bridges and buildings, designing engineer, supervisor of track, division engineer, and engineer of track for the section Buffalo and East.

G. N. EDMONSON, formerly division engineer of the Mohawk Division of the New York Central Lines, succeeds William A. Murray as engineer of track, for the Buffalo and East system. His successor as division engineer of the Mohawk Division is A. R. JONES.

S. E. ARMSTRONG, formerly division engineer of the River Division, New York Central Lines, has been appointed engineer of standards reporting to the engineering assistant to the president of the lines, with office in New York City. S. A. SEELY is appointed division engineer of the Pennsylvania Division, headquarters Jersey Shore, Pa. J. N. GRIM is made division engineer of the Adirondack & Ottawa Division, headquarters Utica, N. Y. J. W. STEVENS is division engineer of the River Division, headquarters Weehawken, N. J. J. H. KELLY is division engineer, Rochester Division, headquarters Rochester, N. Y.

G. C. OLSEN, formerly chief engineer for the St. Joseph Structural Steel Co., St. Joseph, Mo., has moved to Chicago, Ill., and has opened an office for consulting practice in the designing of structural details.

Engineering Societies

Calendar

Annual Meetings

AMERICAN SOCIETY FOR MUNICIPAL IMPROVEMENTS, St. Louis, Mo.; Annual Meeting, Dallas, Texas, Nov. 14-18, 1927.

ASPHALT PAVING CONFERENCE: Under the auspices of the Asphalt Association and Association of Asphalt Paving Technologists; annual conference Atlanta, Ga., Nov. 28-Dec. 2, 1927.

HIGHWAY RESEARCH BOARD, NATIONAL RESEARCH COUNCIL, Washington, D. C.; Annual meeting, Washington, D. C., Dec. 1 and 2, 1927.

INTERNATIONAL ASSOCIATION OF STREET SANITATION OFFICIALS, Chicago, Ill.; Annual Conference, Detroit, Mich., Jan. 9-10, 1928.

AMERICAN ROAD BUILDERS ASSOCIATION, Washington, D. C.; Annual Convention, Cleveland, Ohio, Jan. 9-14, 1928.

AMERICAN SOCIETY OF CIVIL ENGINEERS, New York City; Annual Meeting, New York City, Jan. 18-20, 1928.

ASSOCIATED GENERAL CONTRACTORS, Washington, D. C.; Annual Convention, West Baden, Ind., Jan. 23-27, 1928.

THE SOCIETY OF TERMINAL ENGINEERS at its Nov. 10 meeting in the United Engineering Societies Building, New York City, will have two addresses on the subject, "Labor as a Factor in Terminal Operation and Design," by Albert J. Martin, superintendent of docks, Luckenbach Steamship Co., and Joseph P. Ryan, president, International Longshoremen's Association.

THE MICHIGAN ENGINEERING SOCIETY will hold its annual meeting Nov. 10 to 12 at Jackson, Mich. Its program includes the following papers: "Reinforced-Concrete Structures," William F. Zabriskie, Detroit; "Road Design," C. F Boehler, engineer of design, State Highway Department; "Highway Structures," C. A. Melick, bridge engineer, State Highway Department; "Municipal Engineering and Construction Work," H. C. McClure, city engineer, Flint, Mich.; "Municipal Construction," George R. Cooke, Detroit; "Pollution of Grand River," S. D. Porter, Grand Rapids; "Large Radial-Flow Imhoff Tanks," R. L. McNamee, Ann Arbor; "Sewage-Works Service Charges," Milton P. Adams, city sanitary engineer, Grand Rapids; "Curve Plots," W. B. Williams, Grand Rapids; "Steam Power Plants," J. W. Parker, Detroit Edison Co.; "Developments in Hydro-electric Plants," E. M. Burd, Commonwealth Power Co., Jackson; "High Voltage Transmission Lines," J. G. Hemstreet, Consumers Power Co., Jackson. L. E. Ayres, Ann Arbor, is president of the society; the secretary is E. L. Brandt, Detroit.

THE SAN FRANCISCO SECTION of the American Society of Civil Engineers held a field trip Oct. 29 to witness the sinking of the twelfth and last segment in the Oakland estuary tube. In addition to watching the segment being lowered into place, opportunity was afforded for inspecting the interior of those segments already in place whose joints have been completed under water and the bulkheads removed.

Obituary

OSCAR JONES, a topographic and geodetic engineer with the U. S. Geological Survey, serving at Washington, D. C., died at his home Oct. 21 of ptomaine poisoning contracted while on a field survey at Mifflinton, Pa. He was 58 years of age. Mr. Jones was born in Murfreesboro, Tenn., and graduated in engineering from the University of Tennessee. He had served with the Geological Survey since 1900.

JAMES H. KENNEDY, Vancouver, B. C., pioneer railway builder, died Oct. 22 at his home, aged 76 years. Mr. Kennedy was in the early construction days associated with Sir William Van Horne on construction for the Canadian Pacific Ry. east of Winnipeg, and later with James J. Hill on construction for the Great Northern Ry.

WILLIAM L. SCOTT, Montreal, Que., who was known in railway, harbor and water-works construction, died Nov. 2 at his home, aged 75 years. Mr. Scott was born in Somerville, Mass., and graduated as a civil engineer from Woodstock College. After a period of railway work he was engaged on the Hamilton and Ottawa water-works, and later was appointed assistant engineer on the Montreal ship canal. For many years he worked for the Montreal Harbor Commission. When the Quebec bridge was started he was an assistant engineer on the original pier foundations. In his later years he practiced as a consulting engineer.

JOHN G. ROSE, materials engineer with the federal bureau of public roads, Denver, Colo., died Oct. 27 after an operation for appendicitis. Mr. Rose was a graduate of the University of Colorado.

WILLIAM WHITTEN, Puntagorda, Fla., former city engineer of South Bend, Ind., and former St. Joseph County surveyor, died recently at his home, aged 66 years. Mr. Whitten served as surveyor of St. Joseph County, Ind., for many years, beginning 1865. He moved from South Bend to Florida 22 years ago.

Construction Equipment and Materials

A Section Devoted to What the Manufacturer Is Doing for the Engineer and Contractor

Home Exhibit Devoted to Construction Processes

An exhibit, differing from the great majority of improved home exhibits in that it is devoted to construction processes rather than to the finished building, has been prepared by the members of the National Committee on Wood Utilization of the Department of Commerce for the Washington, D. C., real estate exhibit. Methods of guarding against details that will be hidden in the finished structure and yet cause such defects as plaster cracks, or squeeky and sagging floors are shown. Full sized models show proper methods of framing the lumber parts of homes of wood frame or of wood and masonry construction. The committee believes that a growing recognition of thorough craftsmanship and a greater demand for durable houses will come from a better understanding of good construction; and good construction means that the lumber going into the house renders the maximum service.

Business Notes

DAVENPORT LOCOMOTIVE WORKS, Davenport, Iowa, announces that its manufacturing plant and entire business has been acquired and will hereafter be conducted by the Davenport Locomotive & Manufacturing Corp. The change places the company in a much strengthened financial condition. The company has recently introduced a new gas-electric locomotive in sizes from 4 tons up to 30 tons.

JEFFREY MANUFACTURING CO., Columbus, Ohio, announces the election of Robert W. Gillespie to vice president and general manager. Mr. Gillespie, who for many years was connected with the Bethlehem Steel Co., joined the Jeffrey organization as vice president and assistant general manager on July 1, 1926.

HARNISCHFEGER CORP., Milwaukee Wis., announces the opening of a branch office at Boston, Mass.

J. F. PRETTYMAN & SONS, Charleston, S. C., has recently completed a modern and complete wood preserving plant costing $650,000, and having a normal operating capacity range between 55,000,000 and 110,000,000 ft. b.m. annually. The plant was designed by Grant B. Shipley, Pittsburgh, Pa. A private wharf serves the plant, providing facilities for receiving the creosote oil, and direct connection is afforded with the three railroads serving Charleston. The company owns 60,000 acres of timberland, and operates its own sawmills, insuring a steady source of lumber and timber. Both the full-cell and the empty-cell processes of creosoting will be used.

HARRY HOUGH, for 10 years vice-president and comptroller, B. F. Goodrich Rubber Co., Akron, Ohio, has recently been elected president. Before joining the Goodrich company, Mr. Hough was associated with an accounting firm in New York City. D. M. Goodrich, son of Dr. B. F. Goodrich, founder of the company, was elected chairman of the board of directors.

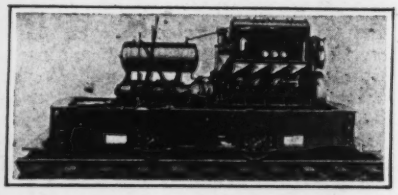
AMERICAN-LAFRANCE FIRE ENGINE CO., INC., Elmira, N. Y,. has acquired the FOAMITE-CHILDS CORP., Utica, N. Y,. and formed a new company to be known as the American-LaFrance and Foamite Corp. The combination of the two companies will make possible the rendering of a broader service in the fire protection engineering field.

FRANK A. PECK, vice-president, Universal Crane Co., Cleveland, and for a number of years director of the Thew Shovel Co., Lorain, Ohio, has recently been made an active vice-president in the latter company. Mr. Peck has been associated with the crane and shovel industry for over 30 years. He will retain his position in both companies, since they are closely affiliated.

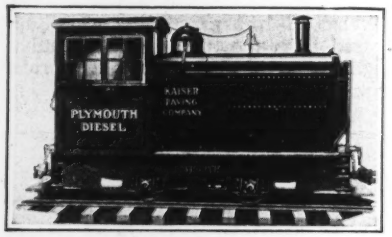
New Developments

Diesel Power Is Introduced On Industrial Locomotives

What is believed by its manufacturers to be the first Diesel-powered, gear-driven locomotive built in America, has just been brought out by the Plymouth Locomotive Works (the Fate-Root-Heath Co.), Plymouth, Ohio. A saving in fuel cost, effected with the Diesel engine, is said to be the main reason for adopting this form of power. The engine consumes from 2½ to 3½ gallons per hour of fuel oil and the consumption of lubricating oil is somewhat less than in a gasoline engine of equal power. Fuel oil costs vary from 3 cents per gallon on the Pacific Coast to 6 or 7 cents in the midwestern states. The weight of the locomotive is 20,000 lb. All of the wheels are drivers, giving a draw bar pull of 5,000 lb. on dry rails at 2½ m.p.h. Additional speeds of 4, 8¼ and 13½ m.p.h. are provided.



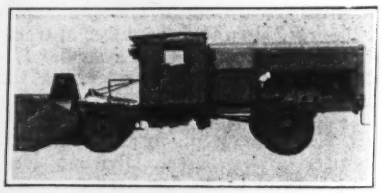
The power plant is a 4-cylinder, 4-cycle, enclosed type Atlas-Imperial full Diesel engine, 6½x8½ in., developing



77 hp. at a speed of 650 r.p.m. Starting is accomplished by compressed air furnished by an air compressor built into the engine. A small combined gas engine and compressor is placed in the cab for emergency use. A sliding gear transmission with four forward speeds and four reverse, is used while the final drive from transmission to driving axles is by means of two short roller chains. A heavy cast-steel frame forms the structure upon which the machinery is mounted, the side frame members having suitable provision for mounting the roller bearing axle boxes. The first 3 of the 10 ton locomotives have been used by the Kaiser Paving Co., Oakland, Calif. These Diesel locomotives will be made in a full range of sizes from 10 tons up to 50 tons, and the larger sizes will soon be in production.

Blower Type Snow Plow Built as Complete Truck Unit

A complete snow removal unit, consisting of truck chassis, auger-and-blower snow plow, and a gasoline engine power unit operating both the

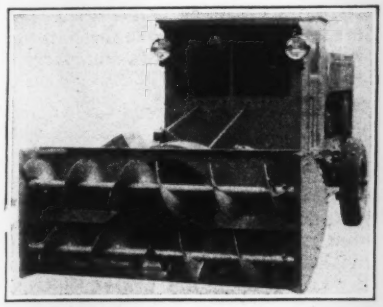


truck and the snow removal unit, has been developed by the Klauer Mfg. Co., Dubuque, Iowa. The accompanying illustrations show a side view of the complete unit, and a front view of the operating hood. The augers are designed to cut into the snow, breaking it up, and moving it to the center of the hood, where, by the forward movement of the machine, it is received by the blower and blown completely off the roadway. Machine propulsion requires very little power, since the

augers cut their own way into the snow. The greater part of the power is required by the blower and is provided by a six-cylinder Climax engine developing 125 hp. at 100 r.p.m.

The front assembly consists of the hood, augers, fan, blower housing, bevel gear transmission housing, clutch, and supporting frame. This entire mechanism, suspended on stub shafts attached to the inside of the frame, can be raised and lowered from the cab, the weight being compensated by heavy extension springs. The sides of the hood are reinforced on the bottom by cast steel shoes, while the snow cutting augers consist of ¼-in. heat treated steel flights, 18 in. in diameter, mounted on 3-in. shafts, running on roller bearings. To safeguard against damage to augers from buried obstructions, a shearing pin has been provided. Striking solid objects will cause the pin to be sheared off, stopping the augers. The pin can be instantly replaced.

The machine has three transmissions, the first acting as a speed reducer for both the plow and the rear wheel drive; the second provides four speeds



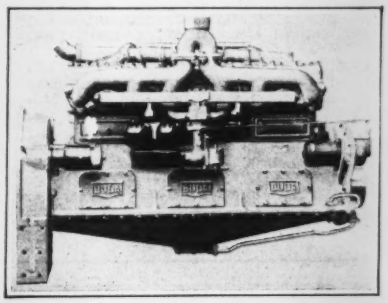
forward and one reverse; and the third is a two-speed transmission, in series with the second transmission, providing additional speed reduction, affording a total selection of eight forward speeds. The slow speed of the machine is approximately ½ m.p.h. A speed of 25 to 30 m.p.h. can be attained when not moving snow. The total weight of the machine is 11,500 lb., its width is 8 ft. overall, and its length overall is 23 ft.

Hook Permits One-Man Operation of Long Steel Tape

An improved hook for steel tapes has recently been announced by the Lufkin Rule Co., Saginaw, Mich., which enables one man to accurately take long measurements unassisted. The hook is easily slipped on and off the tape, yet cannot fall off. The zero reading of the tape falls at the inside of the hook, which has a toothed edge insuring a firm grip the instant tension is applied to the line. The hook swings with the ring of the tape, thus guarding against breakage of the line, and also allowing the hook to fold against the case when the line is wound in. The hook, of composition metal and 2 in. long, is suitable for steel tapes ⅜ in. wide.

Heavy-Duty Gasoline Engine Develops 150 Hp.

A new model heavy duty industrial gasoline engine has recently been developed by the Buda Co., Harvey, Ill., containing many advanced features of design. A patented force-feed lubricating system feeds through a seamless



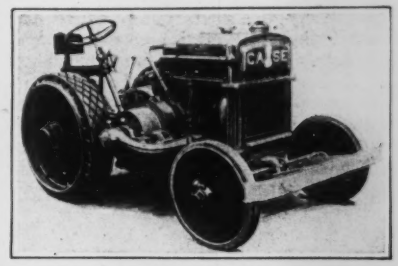
steel tube cast integral with the crankcase to all crankshaft, camshaft and connecting rod bearings. The oil pump is submerged in oil, the sump capacity being 22 quarts. The motor is 6 cylinder, "L" head type, the cylinders having a 6-in. bore and a 7¼-in. stroke. Special attention has been paid in the head design to produce maximum power without detonation. Large water passages are also provided around all valves, and removable valve guides and push rod guides are used.

The crankshaft is of open hearth steel supported on four bearings with bronze shell and babbitt lining. Chrome-vanadium steel is used for the connecting rod and gray iron for the pistons. Three piston rings are provided above the wrist pin and one wiper or oil ring below. The overall dimensions of the engine, height and width, as illustrated are 44⅞ in. and 21½ in. respectively. The net shipping weight is 3,210 lb.

Compactness and Comfort in New Industrial Tractor

A new industrial tractor intended for use in factories, railroad shops, freight terminals, oil fields, lumber yards, and on road building and maintenance work, has recently been announced by the J. I. Case Threshing Machine Co., Racine, Wis. Because much of the work of such a tractor is in close quarters, the machine has been built compact and low, the latter characteristic contributing greatly to its stability. The tractor can go under low hanging obstructions because there are no projecting parts. In addition to these features, the new tractor can turn completely in a radius of 10 ft.

Another feature, given special consideration in design, is the comfort of the driver. A spring mounted seat, with a back rest, takes up road shocks, and, from this seat, the driver commands a good view all around as well as straight ahead. Ease in getting on

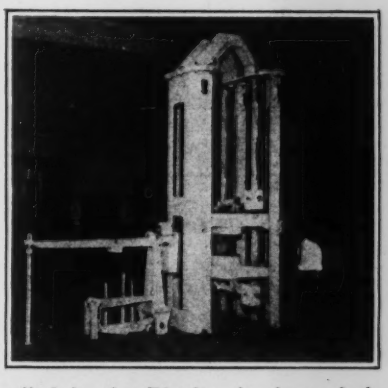


and off the tractor platform is assured because it is built near the ground. Simplicity of operation is also claimed for the tractor. Adjustments are planned to be easily understood and easily made. All necessary controls are within easy reach of the driver and the steering gear is said to move the front wheels quickly without allowing them to jerk out of control.

The weight of the tractor is so divided that approximately two-thirds of it is borne by the rear wheels which are cast hollow, and, when necessary, can be filled with 400 or 500 lb. of sand for additional traction. The tractor is rated at 12 hp. on the drawbar and 20 hp. on the belt while a quick-acting throttling governor regulates the speed of the engine to meet any variations in load.

Large Commercial Testing Machine Installed

A universal three-screw vertical type testing machine with a maximum capacity of 600,000 lb. has recently been in-



stalled in the Pittsburgh plant of the Pittsburgh Testing Laboratory. This is said to be the largest universal type testing machine in commercial service in the United States and Canada. It weighs approximately 30 tons, measures 16 ft. from the top of the columns to the floor line, and has a distance of 40 in. between screws. The machine will take compression specimens 10 ft. 6 in. long and tensile specimens 8 ft. 6 in. long. The gripping jaws of the machine take flat bars in sizes up to 3½ in. thick by 6 in. wide and round bars up to 5 in. in diameter. The machine is said to have been installed because of the growing tendency toward mass testing.

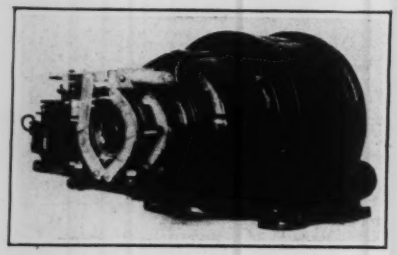
New Contractor's Pump for Road or Well Point Work

A newly designed pump called the Durex, that has all its working parts completely enclosed, is announced by C. I. Longenecker, Milwaukee, Wis. Assurance of constant lubrication is one of the pump's important characteristics, for a separate oil pump supplies oil to the working parts of the pump, while

the 35-hp. Waukesha motor has its own oil supply. A twin-disc clutch is mounted between the engine and pump to permit easier starting. The spring mounting of the pump has a knuckle type front axle, and a spring draw bar with a heavy eyebolt is also furnished. The Durex pump supplies sufficient water for a 27-E paver and can also be used for well point work. Its capacity is 80 g.p.m. at any pressure up to 500 lb.

New Line of Electric Motors For Portable Hoists

A new line of motors for portable electric hoists and other applications involving intermittent operation and high starting torque has recently been announced by the General Electric Co., Schenectady, N. Y. The line runs from 1 to 10 hp. in the direct-current type, and from 1 to 11 hp. single speed, three and two-phase, and from 1 to 5 hp., single phase, in the alternating-current type. Three and two-phase, slip ring



type hoist motors are also included from 3 hp. up. The new motors are totally inclosed, and the sleeve type bearings are waste packed. The direct-current motors are series wound, and the alternating-current motors of the polyphase type have high resistance rotors.

Since many applications require a brake, these motors are all built with shaft extension away from the drive, and the motor end-shield at this end is fitted with pads or bosses with drilled and tapped holes for mounting the brake. The illustration shows the solenoid brake on a direct current type.

Manufacturers and Trade Associations

Calendar

Annual Meetings

NATIONAL EXPOSITION OF POWER AND MECHANICAL ENGINEERING. Sixth Annual Show. New York City, Dec. 5-10.

NATIONAL SAND AND GRAVEL ASSOCIATION, Washington, D. C.; Annual Convention, Detroit, Jan. 4-6, 1928.

ASSOCIATED EQUIPMENT DISTRIBUTORS, Milwaukee; Annual meeting, Cleveland, Jan. 6-7, 1928.

AMERICAN ROAD BUILDERS ASSOCIATION, Washington, D. C.; Annual Convention, Cleveland, Ohio, Jan. 9-14, 1928.

NATIONAL CRUSHED STONE ASSOCIATION, Washington, D. C.; Annual Convention, West Baden, Ind., Jan. 16-19, 1928.

New Publications

Concrete—KOEHRING Co., Milwaukee, Wis., has published a revised edition of its book "Concrete—Its Manufacture and Use." Entirely new tables on required quantities of materials are given and the booklet contains, as before, much useful information. Such chapter headings as "Field Operations in Concrete Construction," "Concrete and Highway Construction," "Miscellaneous Notes for Superintendents and Foremen," "Forms," "Use of Reinforcing Steel," "Estimating," "Waterproofing," "Placing Concrete Under Water," "Care and Operation of Mechanical Equipment," assure any "manufacturer" of concrete that this edition of the book will be as useful as the five previous editions.

Power Shovels—THEW SHOVEL Co., Lorain, Ohio, has published a small booklet explaining the principle of operation of the center drive used on its power shovels and cranes. Diagrams anl illustrations are given and in addition a list of owners of the company's shovels.

Oil-Burner Heaters—ELECTROL INC. OF MISSOURI, St. Louis, Mo., has issued a 29-p. catalog of its different types of Electrol oil-burners, with electric master control, for plants heating buildings by steam, hot water or hot air. The larger sizes are suitable for hospitals, schools, office and apartment buildings, theaters, factories and industrial plants. Other sizes are for large and small residences and for apartment buildings of the average class. A special combination unit of incinerator and hot-water heater is supplied for apartments, hotels, schools and other buildings where large quantities of hot water are heated and a garbage burner or incinerator is also desired.

Air Made Wells—AIR MADE WELL Co., Oklahoma City, Oklahoma, has recently published five new bulletins describing installations of its air made wells using Sullivan air lift equipment in various parts of Kansas, Oklahoma and Texas. One bulletin, No. 56-J, fully describes the wells, their manner of development and the equipment used.

Structural Waterproofing—CONTRACT WATERPROOFING Co., St. Louis, Mo., has published a comprehensive 98-p. catalog describing its Ferro-tite waterproofing. The booklet, after describing the waterproofing, giving the theory of the process and setting forth the method of application, points out its advantages and various uses. The remainder of the catalog, about 75 pages, is devoted to illustrations, geographically grouped of buildings upon which the company has applied its waterproofing material.

Corrugated Culverts—ARMCO CULVERT MFG. ASSOCIATION, Middletown, Ohio, has published a booklet entitled "How Tests Can Explain What Practice Has Proved Regarding the Strength of Armco Corrugated Culverts." The booklet brings together and discusses the different tests which have been made over a number of years on Armco culverts.

Steel Tanks for Railroads—CHICAGO BRIDGE & IRON WORKS, Chicago, has issued a new booklet entitled "Meeting Railroad Requirements with Horton Steel Tanks." The main part of the booklet describes and illustrates conical-bottom and ellipsoidal-bottom tanks. Provisions for sediment removal and cold-weather operation are considered. Elevated steel tanks for general water service, oil storage tanks with Wiggins floating roofs and standpipes are treated briefly.

Concrete Inserts—MIDWEST STEEL AND SUPPLY Co., Bradford, Pa., has issued a pamphlet describing its I-4 inserts for moulding into the walls of reinforced concrete buildings for the attachment of structural steel shelf angles, etc. The pamphlet illustrates the manner of application of the inserts and shows a number of buildings upon which they have been used.

Bucket Loaders—GEORGE HAISS MFG. Co., INC., New York, has devoted its catalog 527 to its heavy duty creeper loaders. In addition to being an illustrated descriptive catalog of the loaders, the booklet also is an announcement for a number of improvements on the machines. Some cost figures are given and a section of the catalog is devoted to a brief résumé of the handling of concrete aggregate for road work, including diagrams for calculating truck cost and the number of trucks required.

Steel Casting Patterns—SIVYER STEEL CASTING Co., Milwaukee, has published a 54 page booklet telling how patterns for its steel castings are made. The booklet goes into considerable detail with regard to the company's practices, and gives in an interesting manner an idea of this manufacturing operation.

Business Side of Construction

Facts and Events that Affect Cost and Volume

Construction Outlook In The South

A report on building activity in New Orleans, just received from the local *Engineering News-Record* correspondent, shows some interesting figures. According to official city records, permits for the year (to Oct. 22) total $12,880,387, and $10,133,908 of real contracts have been let—$26.60 per capita.

Buildings, to the value of $6,500,000, are now under way, including hotels and other large structures, factories, dormitories and churches. This does not include public-works expenditures—such as the $500,000 a year that is being spent on a new water purification plant.

CONSTRUCTION IN NEW ORLEANS

Projects, worth $17,500,000, are proposed for the immediate future, and include a courthouse, union station, several hospitals, schools and theaters. This does not include, however, a $1,500,000 paving job on which bids are now being asked, nor $2,500,000 for levee protection, etc.

The whole state appears to be at work and planning more work. Twenty-three towns have $7,500,000 worth of buildings under way. Proposed construction in thirty towns reaches $5,000,000—including four water-works systems worth $300,000, and four sewer systems worth $400,000.

Construction activity, expected to develop in the near future, in the entire South, is indicated by *Engineering News-Record's* October statistics of proposed work not yet out for estimate, as follows:

	1927	1926
Virginia	$12,690 000	$800.000
W. Va.	1,647 000	1,548,000
N. C.	1.750 000	4,640,000
Ga.	7,593.000	1,450 000
Fla.	750.000	1,500 000
Ala.	15.000	1,905 000
Miss.	935 000	3.197,000
La.	1,515.000	2.480 000
Ky.	1,000 000	1.425.000
Tenn.	5,463,000	2.995,000
SOUTH	$33,358,000	$21,218,000

Mississippi, concludes the correspondent, is doing $8,000,000 worth of construction in twenty-five towns, and contemplates $9,000,000 worth for thirty towns.

More than $200,000,000 worth of actual contracts for heavy construction have been awarded in the South in the first three quarters of 1927.

Rent Rises Since 1913 in Twenty Cities

A SURVEY of rent increases since 1913 in a number of cities indicates a general rise in keeping with the rise in construction costs. The figures were gathered by *Engineering News-Record's* Construction News correspondents, who were instructed not to select properties whose land values have greatly increased. In some instances, as noted, the rent increases are influenced by higher land values, greater population or increased business; but in general the per cent rises reflect higher costs.

In the twenty cities surveyed and listed in the table the smallest advances occurred in Duluth and Galesburg, Ill., for residential; in Des Moines and St. Louis for office space; and in Columbus and Cleveland for factory space. Birmingham, Charleston, W. Va., and Indianapolis, Ind., report the greatest advances. The average per cent rise since 1913 for the twenty cities is 80 for residential and 95 for offices. Following are explanatory notes covering conditions in some cities:

Akron, Ohio — Population has increased from 40,000 in 1913 to 200,000 in 1927.

Baltimore, Md.—Rents in high-class locations are holding at peak rates of 1920 to 1922, with rents in converted apartment houses declining slowly since 1923.

Chicago, Ill. — Land values in business districts about five times the values in 1913. Average office rental was $2.74 in 1926. Factory rents were 15 to 18 cents per sq.ft. in 1913, and 30 to 35 cents in spring of 1926, and 25 to 30 cents at present. Rents for apartments in good neighborhoods were around $10 per room in 1913, and $25 to $30 at the peak, and $20 at present. While the trend for the city as a whole is downward, real estate valuations in the apartment-house district will be 44 per cent higher this year than last, meaning higher taxes. Correspondent estimates that in Spring of 1928 there will be available sufficient office space for next ten years.

Cleveland, Ohio—Dwelling and apartment rents rose to a peak in 1924 of 100 per cent above 1913, and have since fallen off 20 per cent.

London, Ont.—Rents are lower than in 1913. Residence in good district $10 per room per month in 1913 and in 1927; in poorer district, $5, against $7 fourteen years ago. Office space, $1.25 to $2 per sq.ft., compared with $1.50 to $2.75 in 1913. Factory space, 18 to 24 cents per sq.ft., compared with 48 cents in 1913. No office buildings or factories erected since 1914 except for owners, and no houses built for renting.

Louisville, Ky. — Increases noted in the table are due more to increased population (from 225,000 in 1913 to 335,000 in 1927) and to increased business than to higher building costs.

Memphis, Tenn. — Apartments that rented for $7.50 to $12 per room in 1913 are now $12.50 to $18; dwellings that were $6.25 to $8 per room are now $8.75 to $12.50. First-class office space has advanced 50 to 70 per cent; second-class, 25 per cent.

New York, N. Y.—Apartments that rented for $300 per room per year now bring $600. Cost to build has increased on this class of building from 30 to 40 cents per cu.ft. in 1913 to 65 to 80 cents in 1927. The same advance is noted for offices and lofts.

Richmond, Va.—Definite decreases in rents for dwellings in September, 1927.

St. Louis, Mo.—Rises from 1913 have been as follows: dwellings and flats, $5 per room to $7.50; apartments, $8.75 to $15; offices, $1.30 per sq.ft. to $2; factory, 15 cents to 45 cents per sq.ft.; lofts, up 50 per cent.

West Palm Beach, Fla. — Residence rentals up 200 to 300 per cent, and commercial up 300 to 500 per cent, due chiefly to increases in land values of 300 to 500 per cent.

The following table represents an alphabetical arrangement of the twenty cities covered by the reports received from *E. N.-R.* correspondents. Columbus, O., showed the heaviest increase in rents of residential buildings; Charleston, W. Va., in office structures; and St. Louis, Mo., in factories:

PER CENT RISE IN RENTS SINCE 1913
Engineering News-Record Statistics

	Residential	Office	Loft or Factory
Akron, Ohio	100	100	...
Baltimore, Md.	80	100	50
Birmingham, Ala.	100	100–150	75
Charleston, W. Va.	120	180–200	...
Chicago, Ill.	100	60	67
Cleveland, Ohio	80	140	27
Columbus, Ohio	150	85	20
Des Moines, Iowa	50	33	50
Duluth, Minn.	38	...	47
Galesburg, Ill.	38	70	...
Indianapolis	140	125	...
Kansas City, Mo.	60–70	100–120	25–60
Louisville, Ky.	85	100	90
Memphis, Tenn.	50	50	...
Nashville, Tenn.	50–60	75–100	100
New York, N. Y.	100	100	100
Philadelphia	75	35	80
Richmond, Va.	100	80	slight
St. Louis, Mo.	60	53	50 or 200*
Trenton, N. J.	70	...	50–60

* Lofts up 50 per cent; factories up, 200 per cent.

This Week's Contracts—With Comparisons

Minimum costs observed are: $15,000 for water-works and excavation, drainage, irrigation, levee, river and harbor projects; $25,000 for other public works; $40,000 for industrial and $150,000 for commercial, educational, institutional, religious and other buildings.

Week ended	Public Work	Private Work	Total U. S.
November 10, 1927.	$11,531,000	$71,368,000	$82,899,000
November 3, 1927.	23,400,000	53,055,000	76,455,000
November 11, 1926.	14,843,000	54,965,000	69,808,000
Jan. 1 to date			
1927	1,114,976,000	1,662,978,000	2,777,954,000
1926	962,217,000	1,575,548,000	2,537,765,000

E.N.-R. Index Numbers

On November 1, 1927

COST	VOLUME
201.98	269
Last Year 210.80	Last Year 244

The Index Numbers are published in detail and with charts in the first issue of each month. The Cost Index is given in full from 1903 through 1926, with a thorough explanation of its method of computation, and a graphic comparison with many other index numbers, in the 48-page pamphlet entitled Engineering News-Record Construction Costs.

Weekly Construction Market

	New York	Atlanta	Dallas	Chicago	Minneapolis	Denver	San Francisco	Seattle	Montreal
Steel Products									
Structural shapes, per 100 lb....	$3.34	$3.75	$4.00	$3.10	$3.35	$3.87½	$3.00	$3.00	$4.00
Structural rivets, per 100 lb....	5.00	4.00	4.75	3.60	3.85	4.64	5.50	5.25	5.50
Reinforcing bars, per 100 lb....	3.24	2.99	2.75	2.50	2.87½	3.77½	2.95	3.00	3.57
Steel pipe, black, 2½ to 6 in., lap weld, discount from list	48%	40%	54%	51%	48%	41%	42@53.8%	48%	.....
Cast-iron pipe, 6 in. and over, per net ton	40.60	34.50	52.00	36.20	40.50	52.00	39.00	53.00	39.00
Concreting Material									
Cement, without bags, per bbl. .	2.35†	+2.45	2.05	2.05	2.22	2.85	2.51	2.65	1.41
Gravel, ¾ in., per cu.yd.....	1.75	2.20*	2.38	2.00	1.25*	1.90	1.80	1.25	1.90*
Sand, per cu.yd.	1.00	1.75	2.00	2.00	1.25†	1.00	1.40	1.25	1.35*
Crushed stone, ¾ in., per cu.yd...	1.85	2.50*	2.83	2.00	1.25*	2.50	1.70	2.50	2.00*†
Miscellaneous									
Fir, 3x12 to 12x12, 20 ft. and under, per M.ft. b.m.	.....	.....	.....	39.50(S4S)	36.75 @38.75	34.25	27.00	24.00	50.00†
Pine, 3x12 to 12x12, 20 ft. and under, per M.ft. b.m.....	—60@61†	34.00	56.00†	.....	.....	.....	.....	.....	.....
Lime, finishing, hydrated, ton...	19.45	—21.50	19.00	20.00†	25.50	24.00	27.50	24.00	21.00
Lime, common, lump, per bbl ..	2.35@2.50‡	—1.25	1.82	1.50†	1.70	2.70‡	1.70	2.80‡	10.00*
Common brick, per M	16.00@18.00†	—11.50	13.60	12.00†	13.75†	8@12	14.00	14.00	20.25
Hollow building tile, 4x12x12, per block	Not used	—.068†	.103	.....	.072	.085	..	.10	.10
Hollow partition tile, 4x12x12, per block	—.0966	—.068†	.103	.076†	.072	.085	.108	.09	.08
Linseed oil, raw, 5 bbl. lots, per 7½-lb. gal.	—.77¼	—.83¼	.85	+.80	— 84	.88	.84	.98	.93
Common Labor									
Common labor, union, per hour.	.90⅜	.30	.....	.90	.....	.....	—.50@.60	.70	.....
Common labor, non-union, hour.	.....	.25	30@.50	.82½	.45@.60	.31¼@.50	....	.62½	30@.35

*Ton. †Delivered to job. ‡280-lb. net. +Advance. —Decline.

DEMAND for building materials, as a group, is well in advance of a year ago, according to *E. N.-R.* contract totals covering the entire country. All basic materials taken together, however, show a steadily declining price trend.

The current price movements, with few exceptions, appear to follow transportation requirements as estimated for the fourth quarter. The increase or decrease in number of freight cars needed for the current quarter, by the thirteen Shippers' Regional Advisory Boards of the United States, is estimated as follows: Increases—clay, gravel, sand and stone, 2.6 per cent; cement, 3.9 per cent; lime and plaster, 0.6 per cent; decreases—lumber and forest products, 2.6 per cent; iron and steel, 3.7 per cent; brick and clay products, 4.2 per cent.

Transportation requirements for the fourth quarter, estimated by the Atlantic States Shippers' Advisory Board, are as follows: Increases—cement, 7 per cent; crushed stone, 6.5 per cent; sand and gravel, 1.0 per cent; paints, oils and varnishes, 5 per cent; decreases—brick, hollow tile, and slag, 10 per cent. The following materials show car requirements equal to those of the fourth quarter last year: Terra cotta, rope, iron and steel, lime, gypsum, lumber, prepared roofing and roofing materials, and slate.

Compared with Oct. 27 levels, prices are easier for lumber and clay products in most of the important centers. Lime declined sharply in the Atlanta district during the last two weeks. Cement prices remain firm at all points listed. Linseed oil advanced 1c. per 7½-lb. gal. at Chicago, over the week end.

The Duluth flaxseed market, up to Oct. 29, was below Argentine quotations for this material. Linseed oil prices are, for this reason, comparatively low, yet firm at current levels.

Lumber stocks in the Minneapolis-St. Paul yards, are below normal for this season of the year. Demand, in this district, is sufficiently slow, to keep prices at levels of Oct. 27, 1927.

THIS limited price list is published weekly except in the first issue of each month and gives current prices on the principal construction materials in the chief cities. Valuable suggestions on costs of work can be had by noting actual biddings as reported in the Construction News and Unit Prices sections following.

The first issue of each month carries complete quotations for all of the basic materials and for the rest of the important cities, also wage rates for the principal building trades and common labor. The last complete list will be found in the issue of Nov. 3, the next on Dec. 1. Explanation of prices contained in the table will appear from time to time in this section; last shown Aug. 11, 1927, p. 247.